CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

JUNE 1960 VOL. LV. NO. 6

FIFTY-FIFTH YEAR OF PUBLICATION

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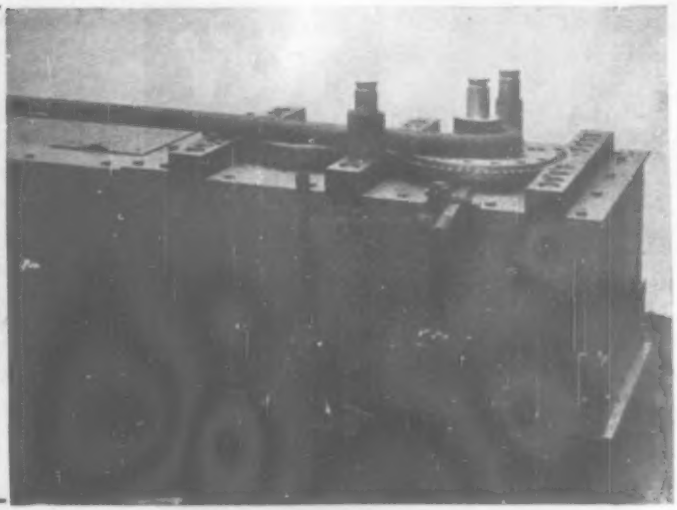

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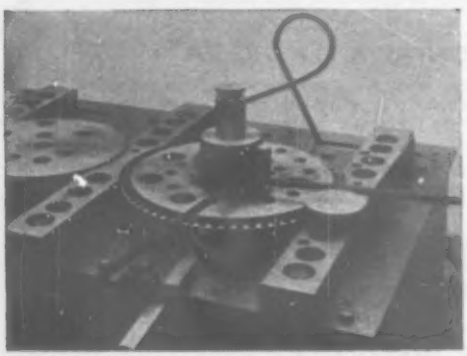

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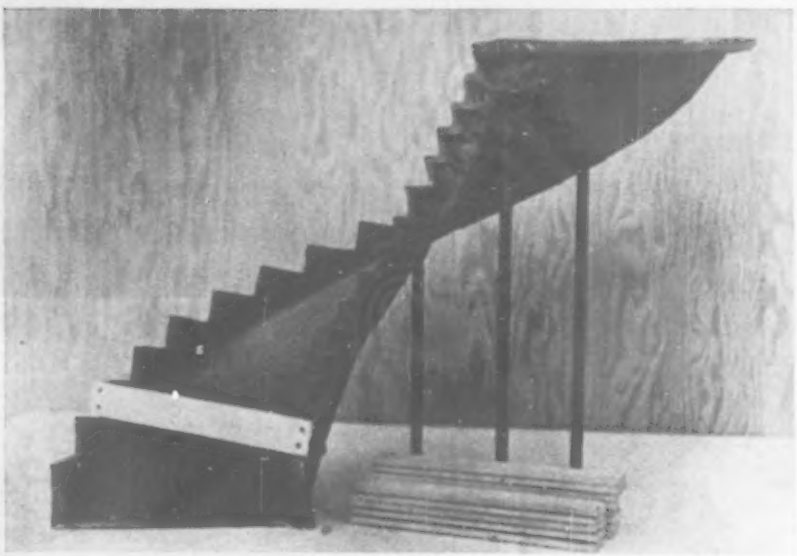

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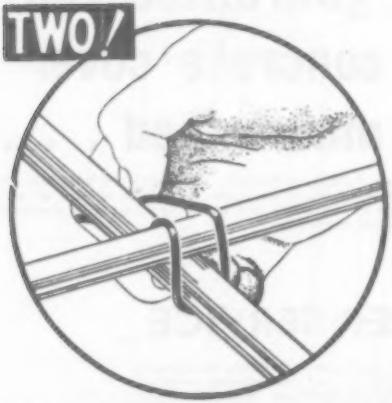

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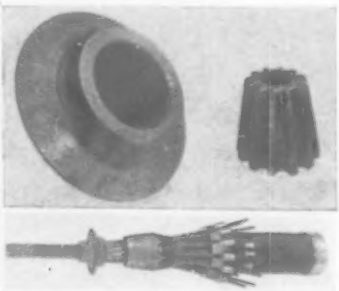

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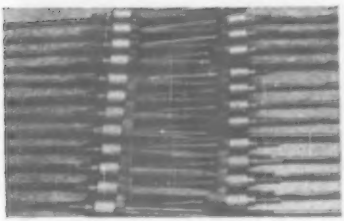

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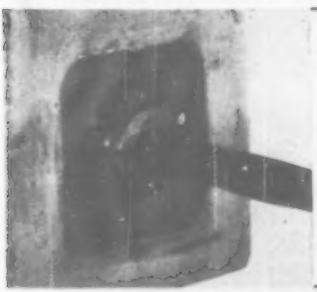

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The P.S.C. MonoStrand system is similar in principle to the MonoWire system in that the anchorage jaws wedge in a conical hole. Two types of MonoStrand anchorage have been developed for use internally or externally. The type illustrated comprises a steel truncated guide cast into the end of a concrete member and the strand anchored by means of serrated steel jaws. External bushes are available for use with bearing plates. Anchorages have been developed for use with all sizes of strand up to and including ½" diameter.

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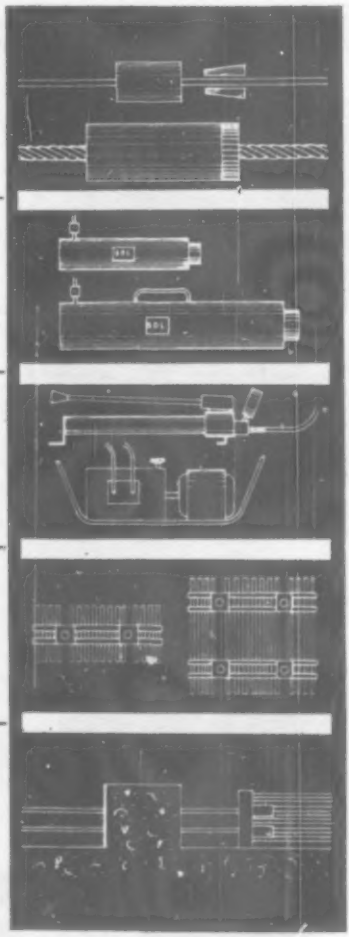

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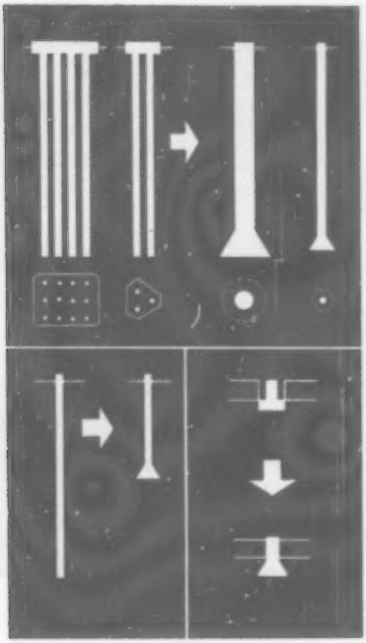

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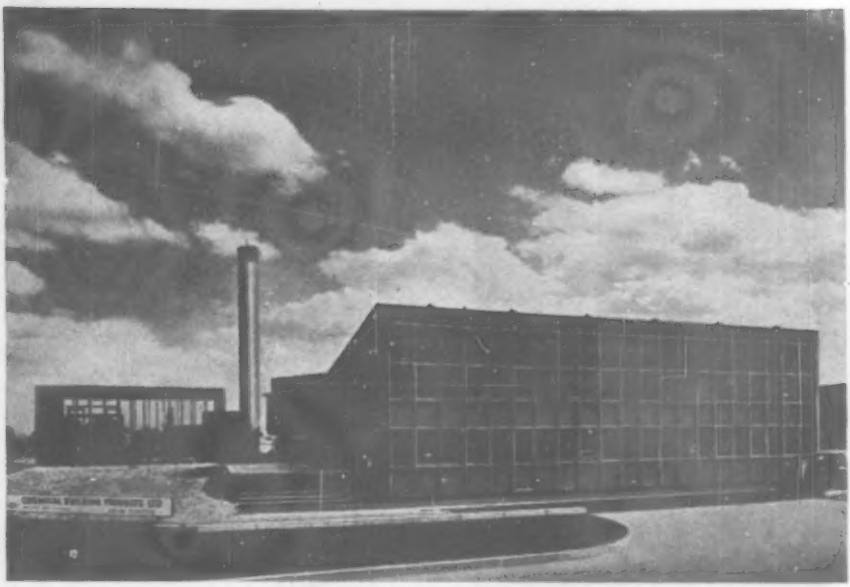

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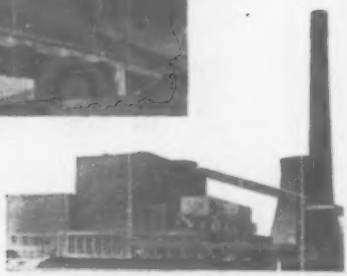

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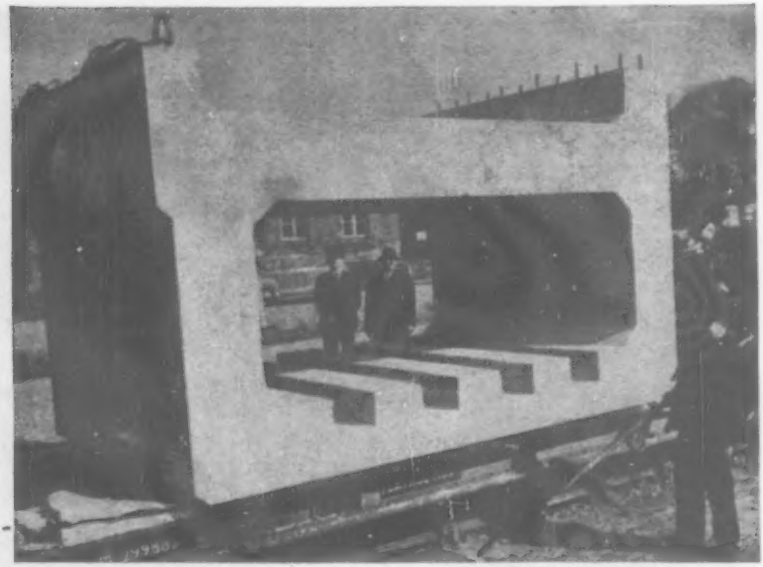

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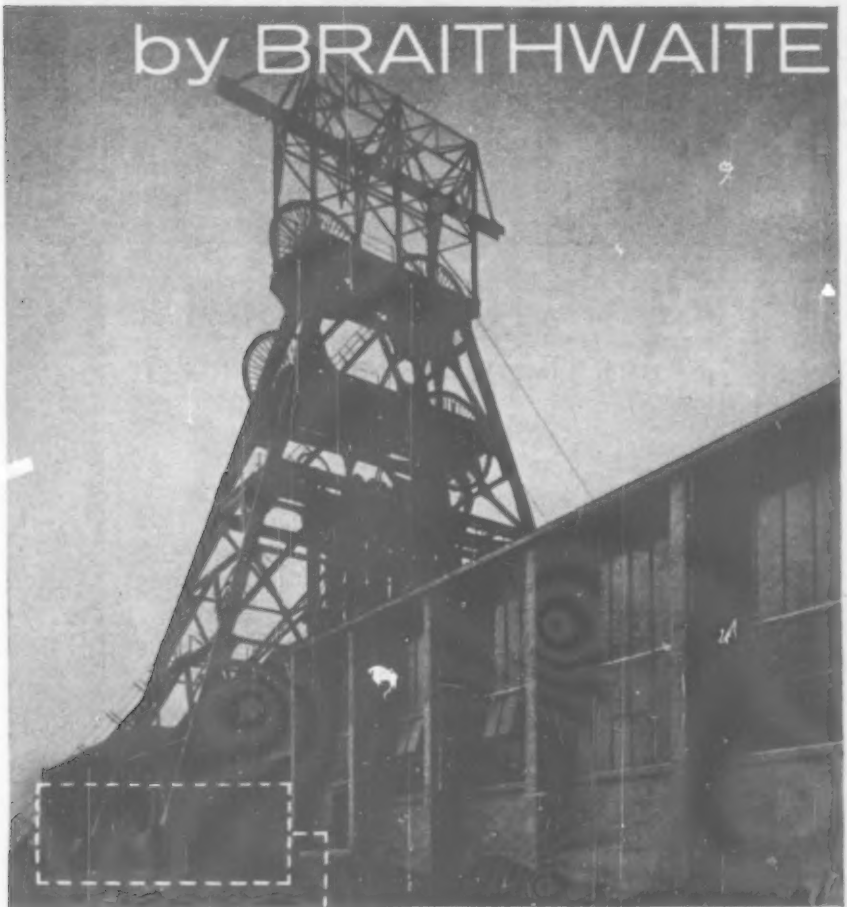

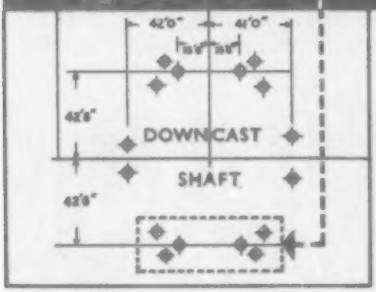

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The pithead gear for Lea Hall Colliery, Rugeley, Staffordshire, is supported on 47 Braithwaite bored cylinders 40" in diameter. The inset shows the pile plan for the downcast shaft, a similar arrangement supports the superstructure for the up-cast shaft and 10 bored piles provide foundations for the winder house, in all 2730 feet of foundation cylinders were sunk.

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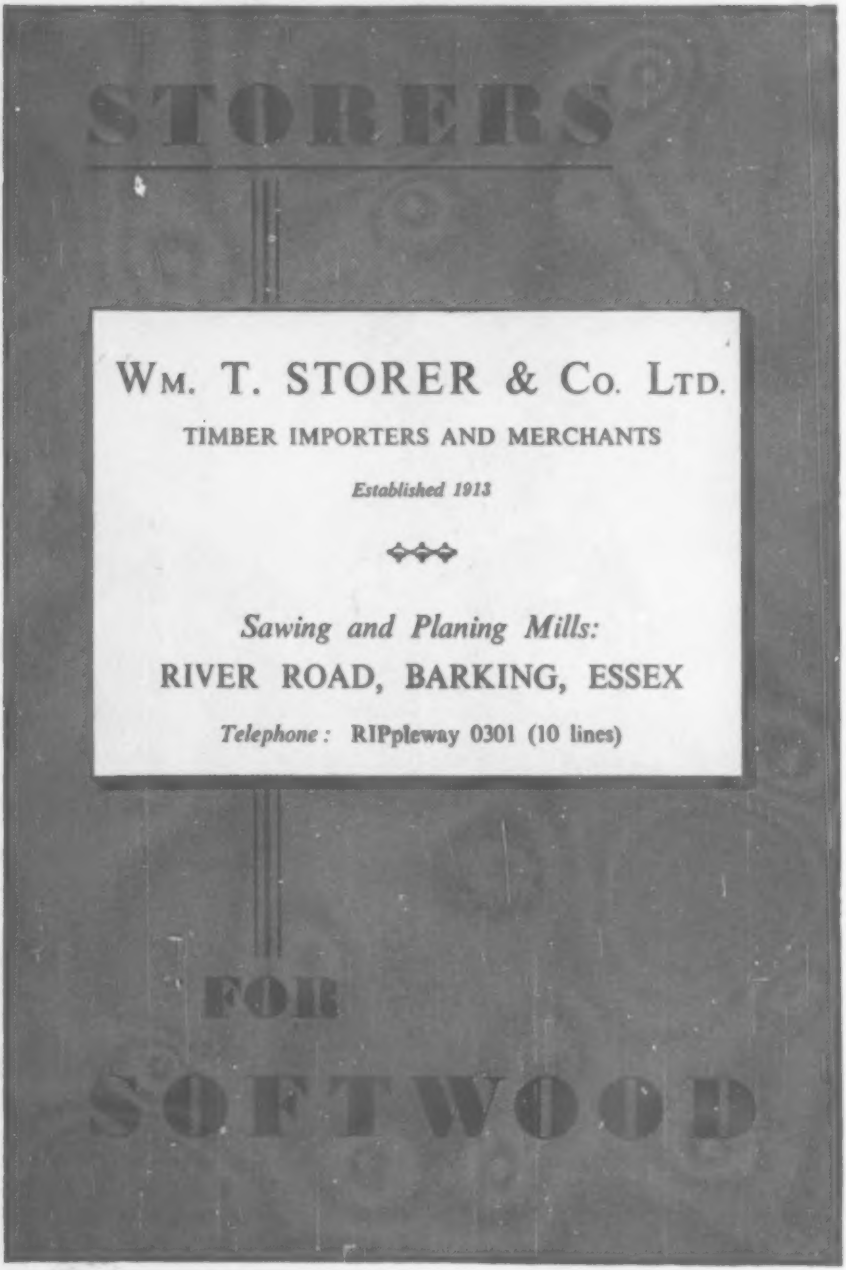
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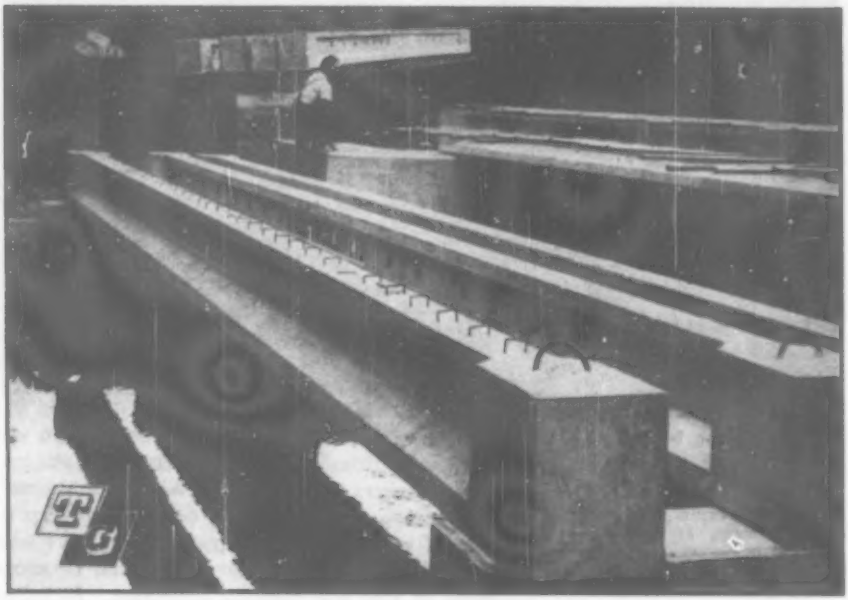

New road bridge during construction on A.50 at Fostons, Derbyshire, showing 40-ft. prestressed beams (pretensioned steel) designed to M.O.T. requirements. Main Contractors: D. T. Bullock & Co. Ltd., Aldridge, Staffs. By courtesy of the County Surveyor of Derbyshire—S. Mehew, Esq., O.B.E., B.Sc., A.M.I.C.E., A.M.T.P.I.

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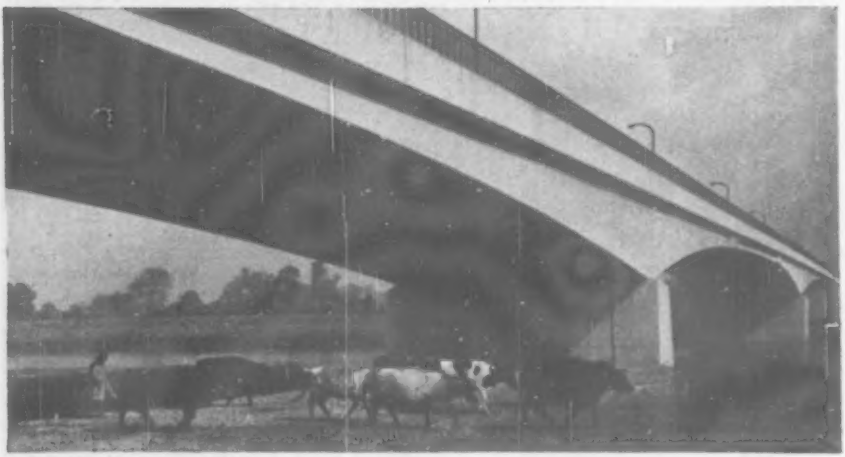

The new Clifton bridge has an overall width of 46 ft. 3 in. and its total continuous length, excluding the northern viaducts, is 800 ft. The centre span directly over the River Trent is 275 ft. long, with a span of 125 ft. on either side. Prestressing was by the Magnel Blaton system.

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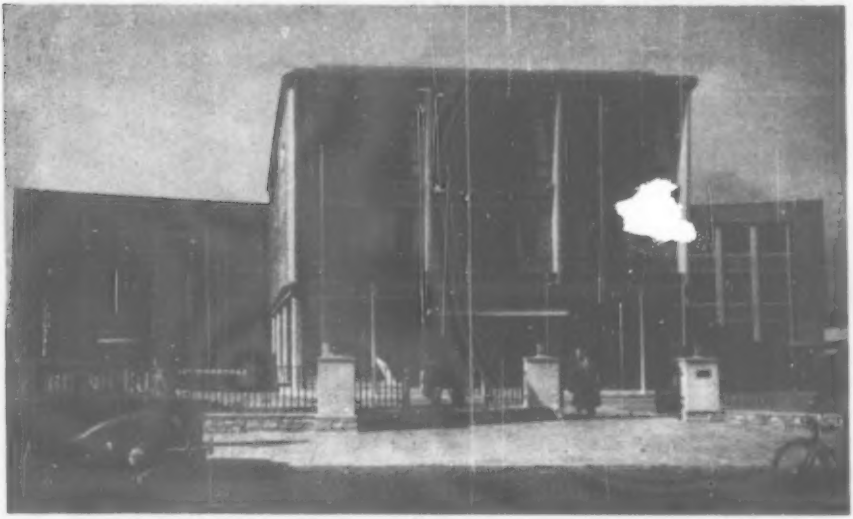

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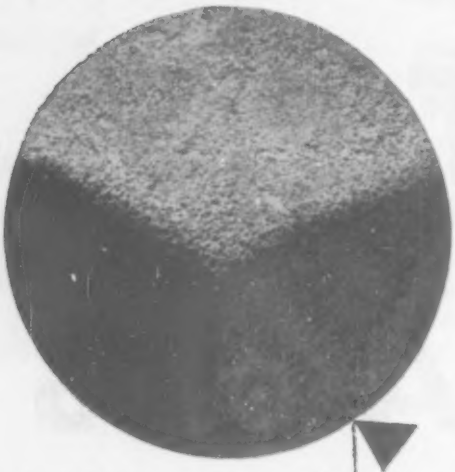

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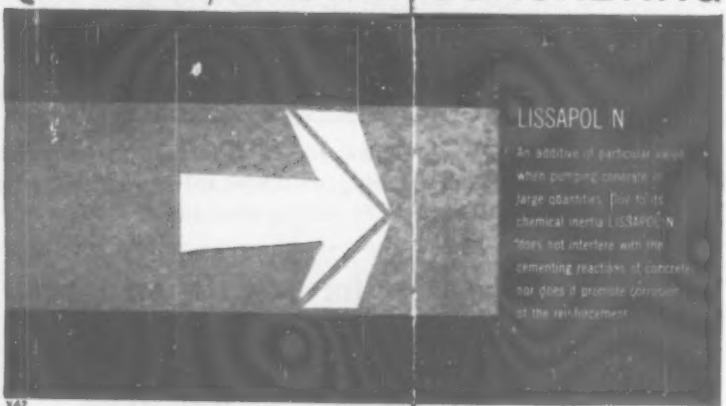

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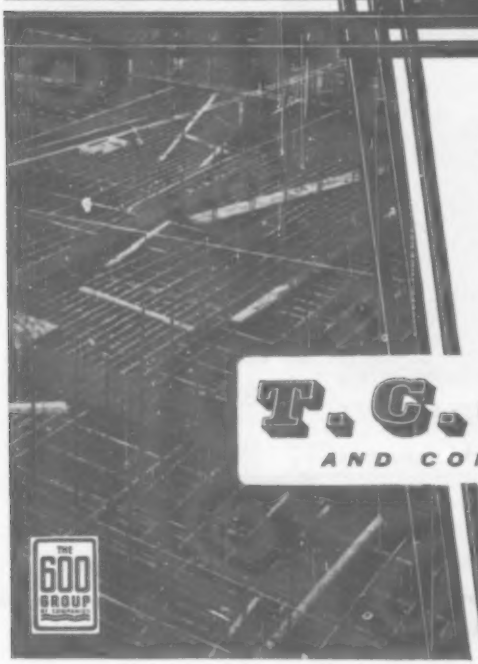

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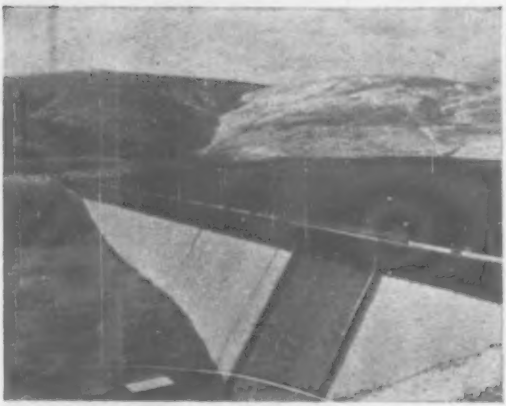

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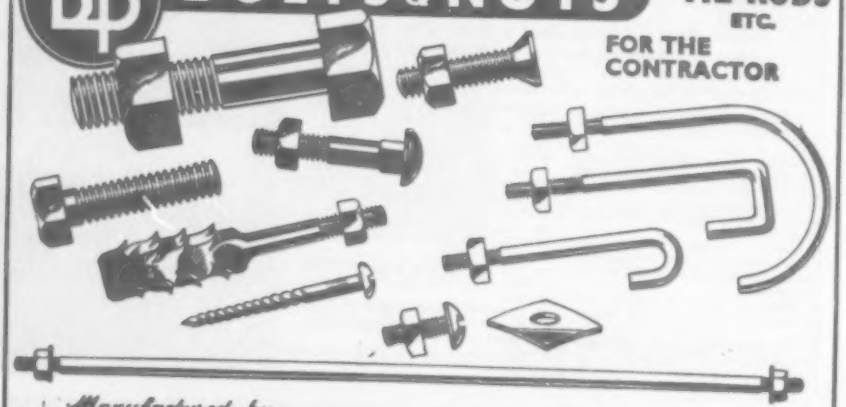

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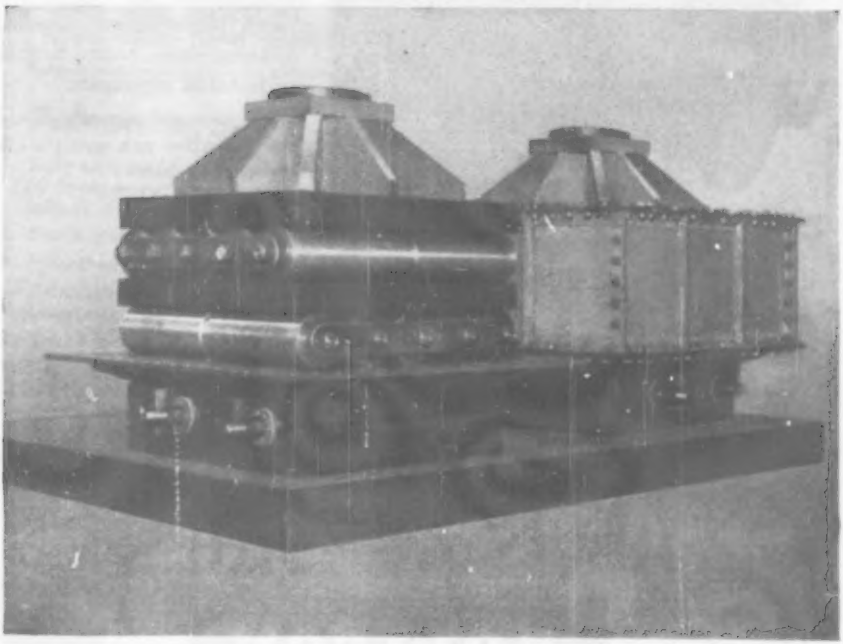

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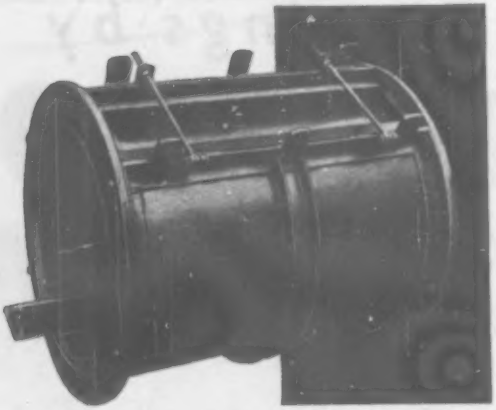

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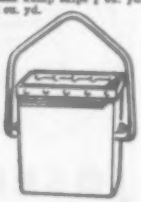

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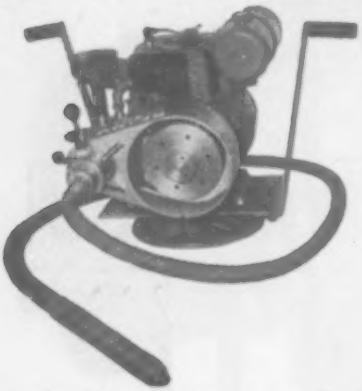

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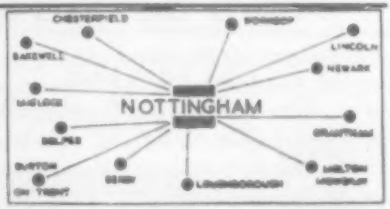

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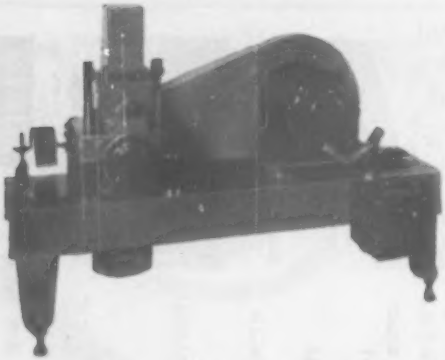

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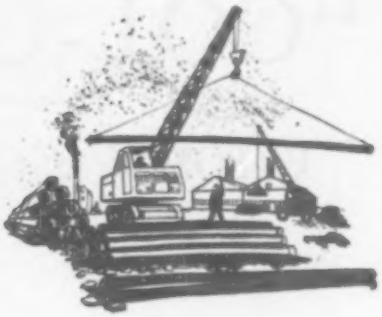

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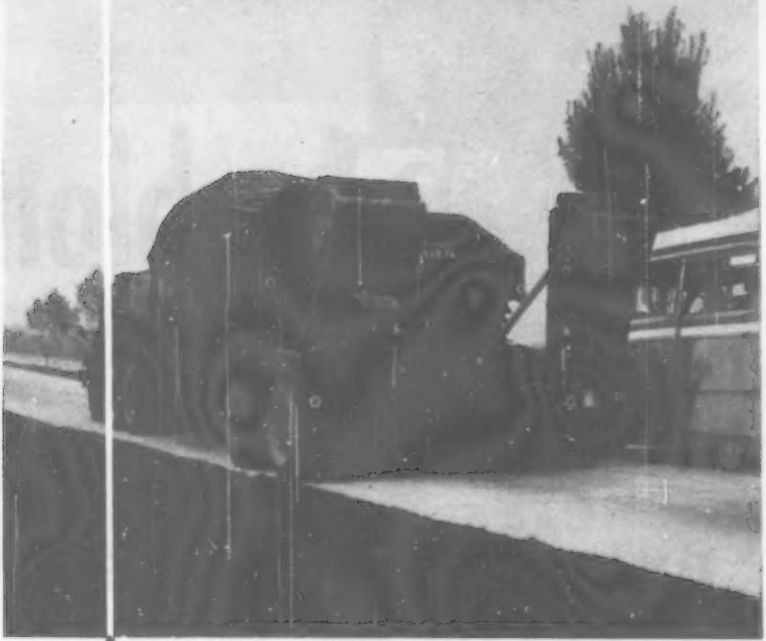

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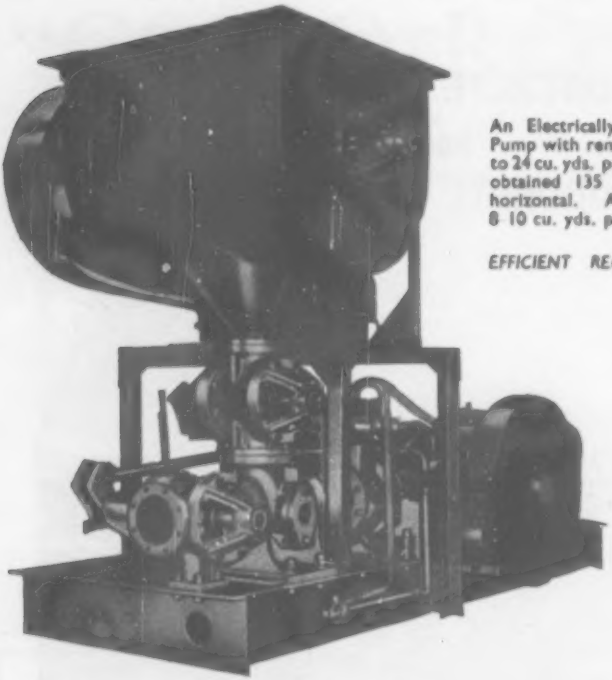

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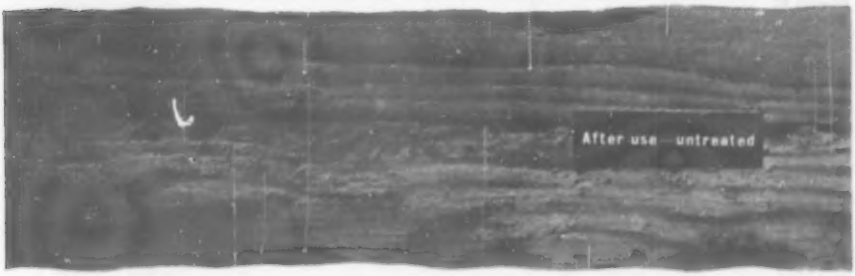

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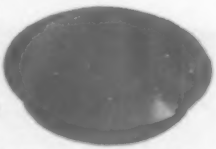

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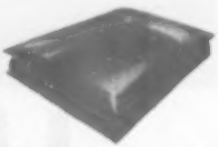

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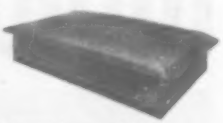

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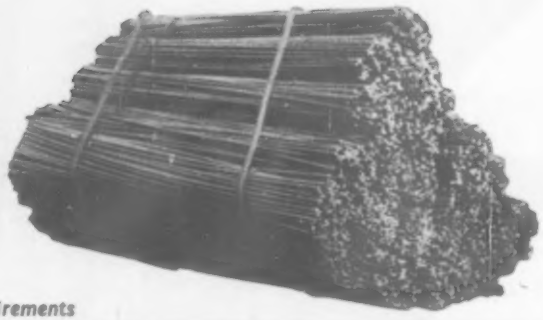

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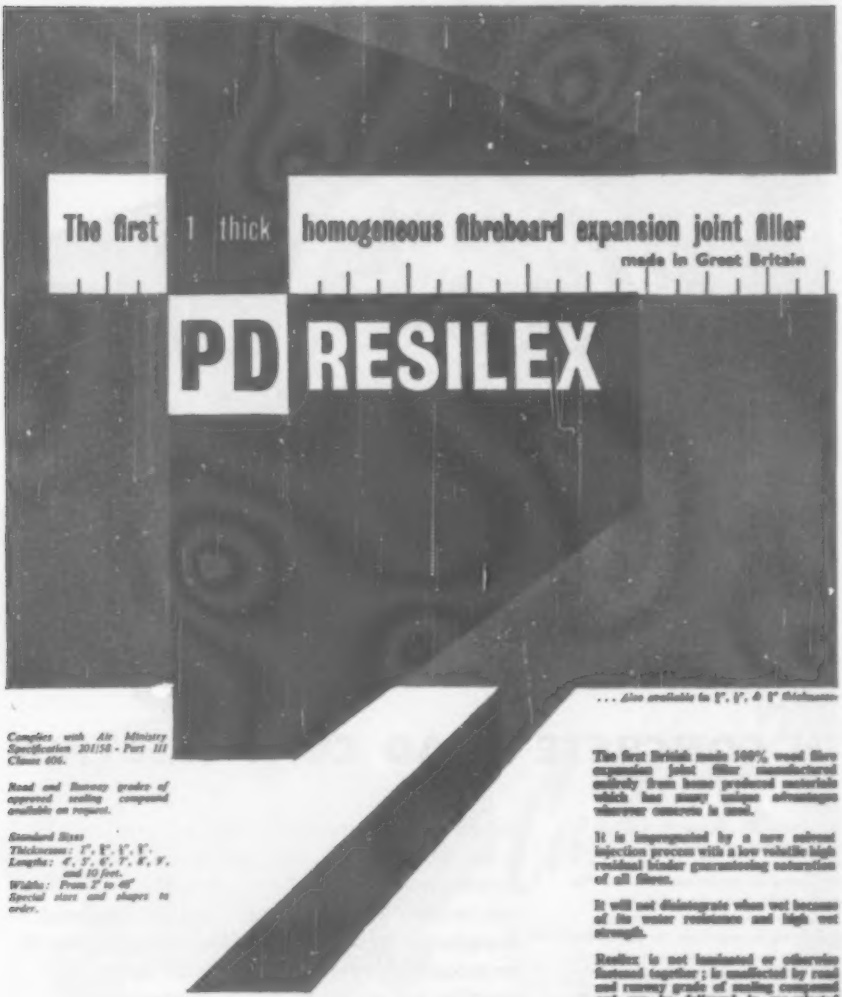

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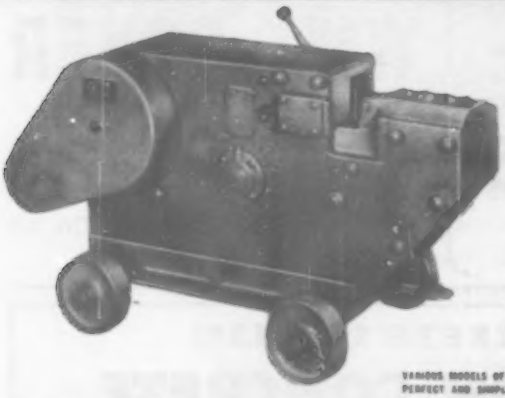

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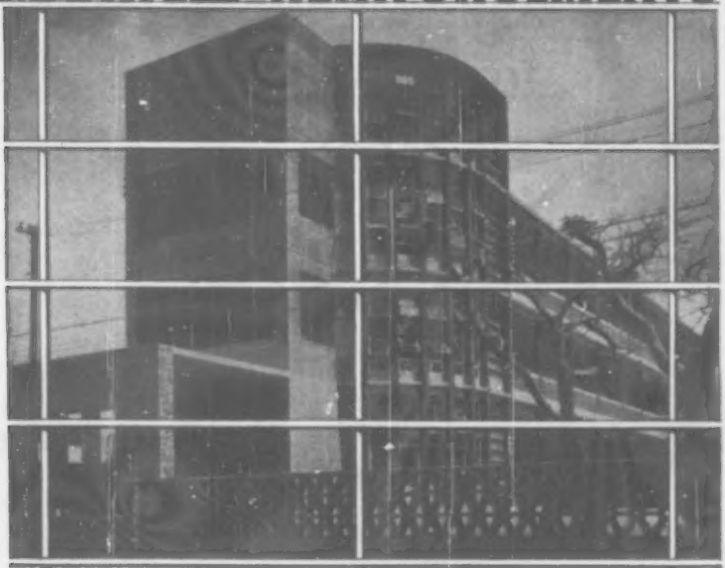

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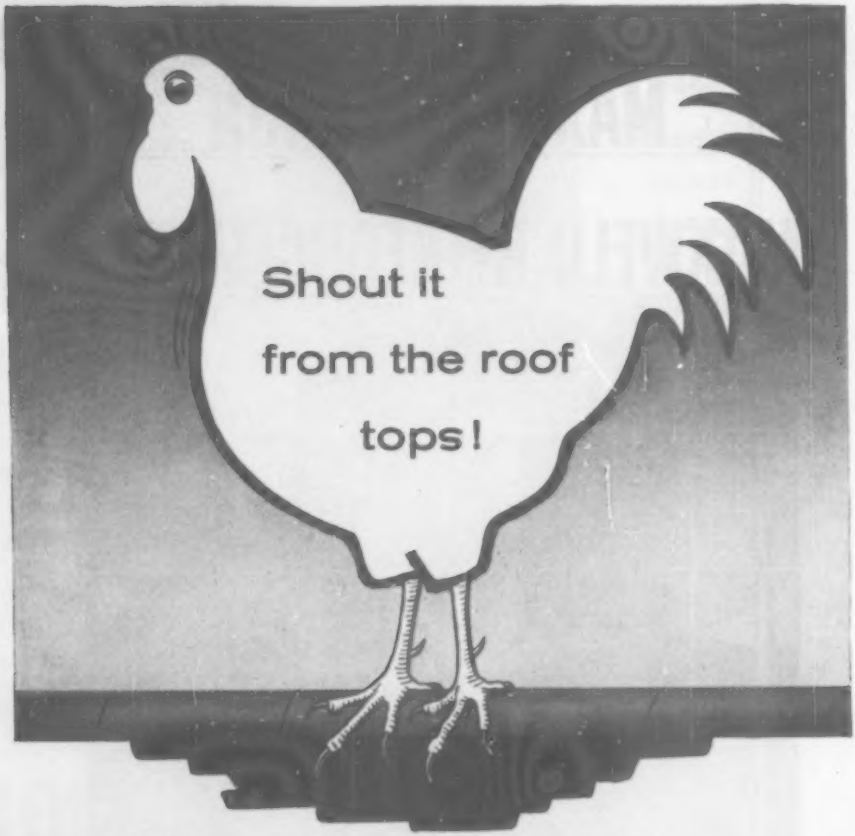

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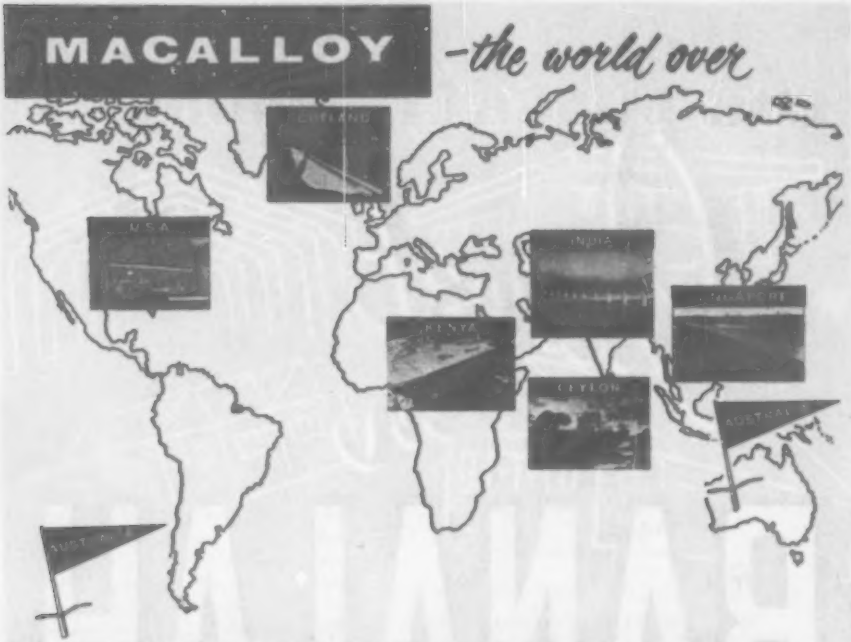

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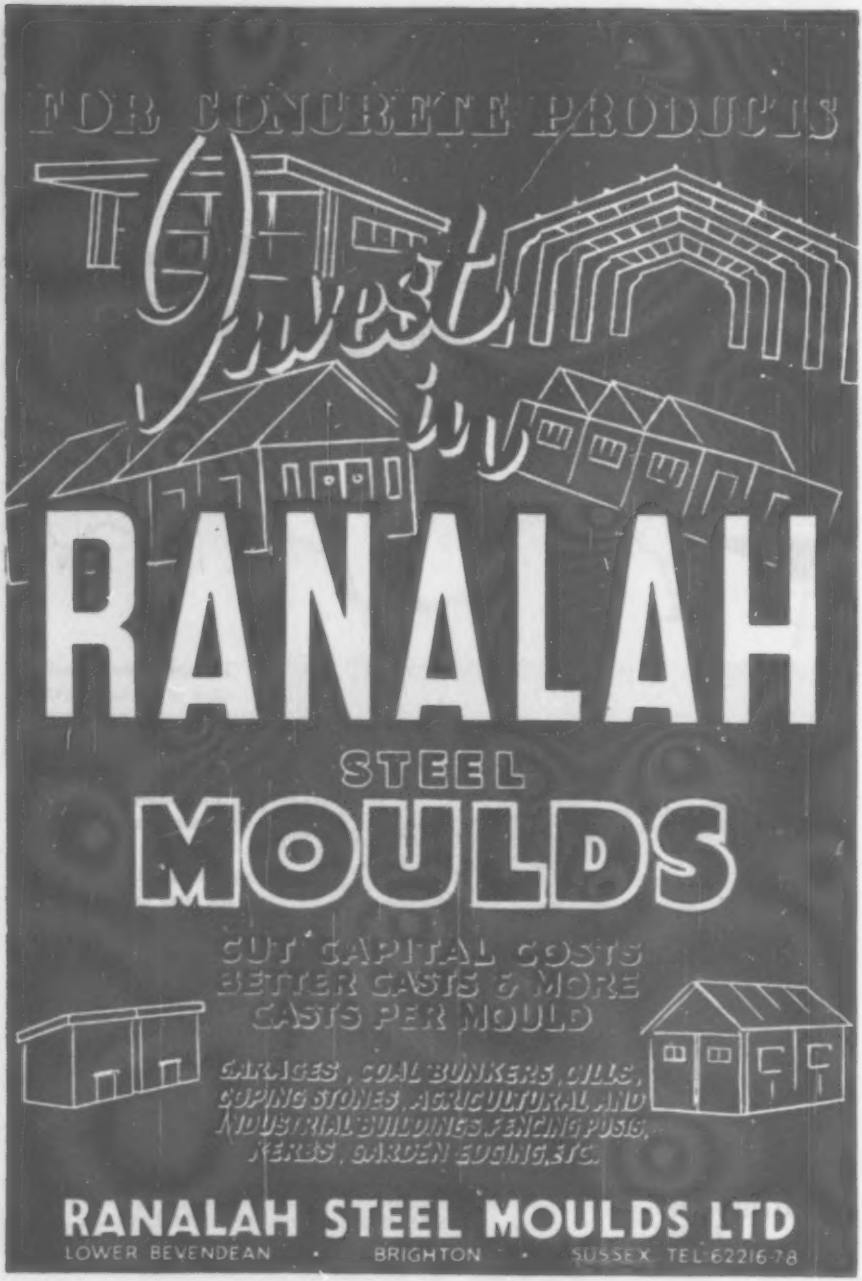
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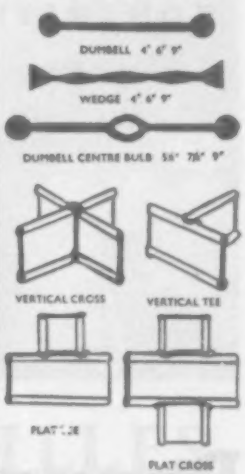

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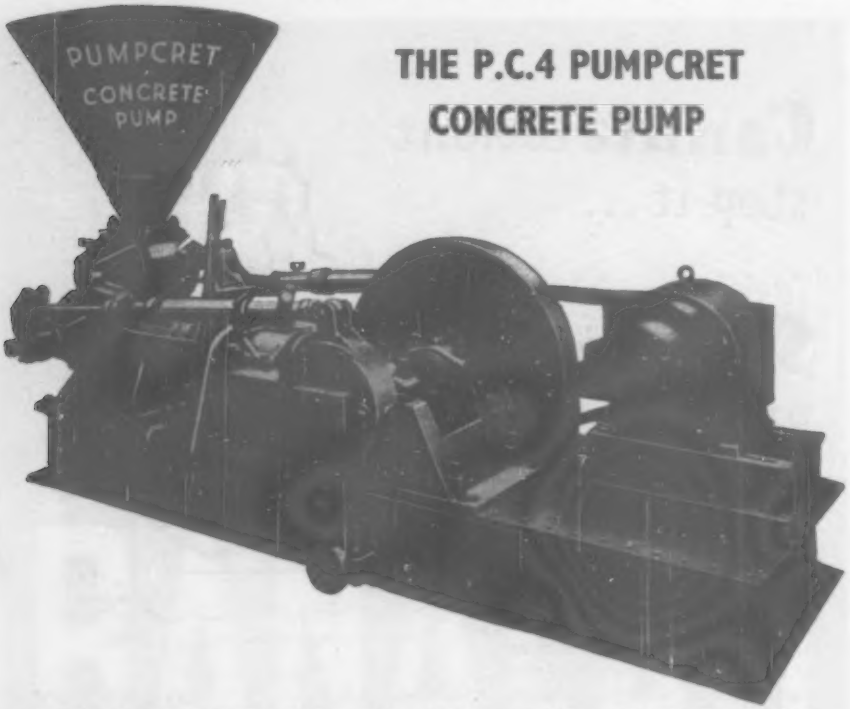

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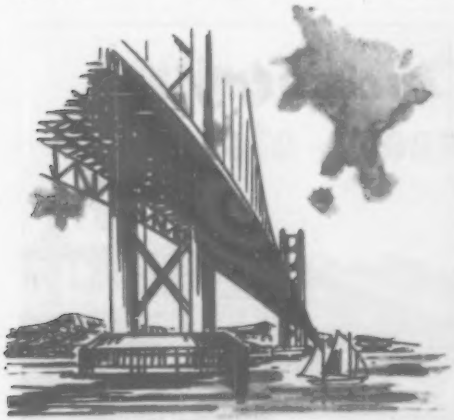

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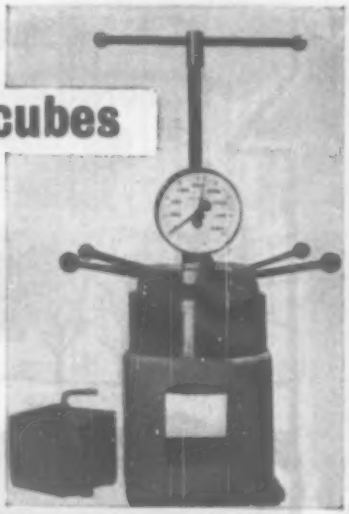

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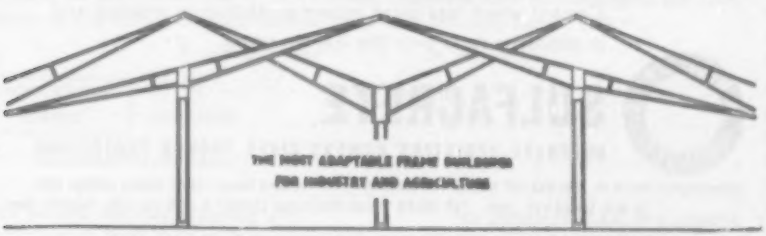

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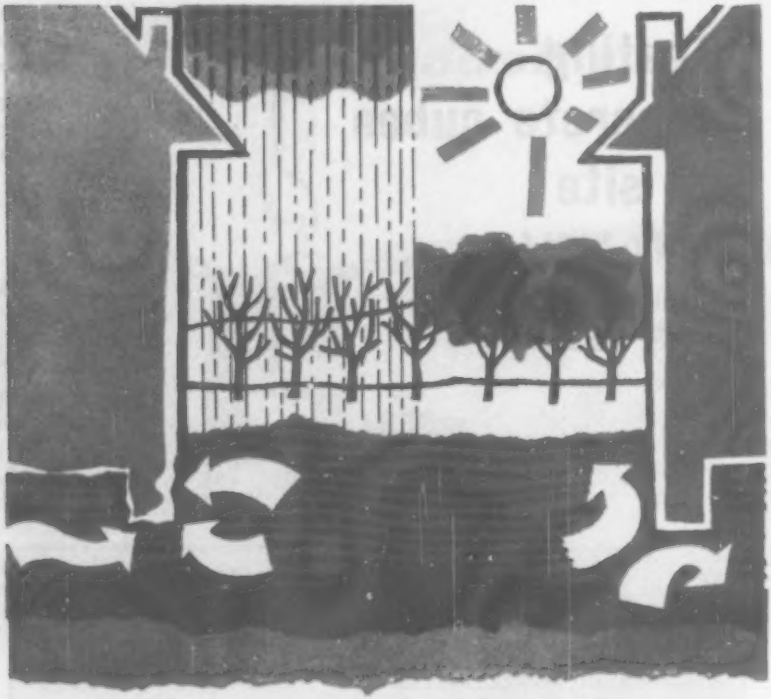

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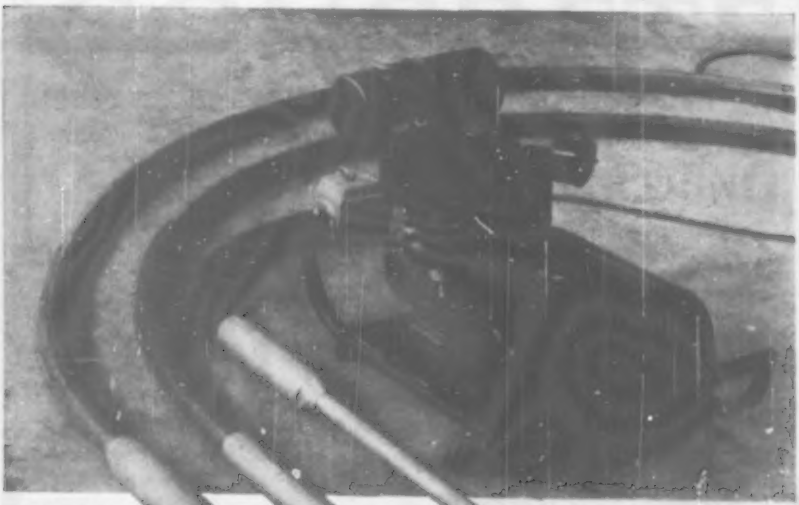

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LV, No. 6. LONDON, JUNE, 1960.

EDITORIAL NOTES

A Code for Reservoirs.

IT is almost ten years since comment was made in these Notes on the revision made in 1950 to the "Code of Practice for the Design and Construction of Reinforced Concrete Structures for the Storage of Liquids", which was issued by the Institution of Civil Engineers. This Code is now superseded by, but constitutes the basis of, a British Standard Code* for reinforced and prestressed concrete structures for the storage of water. Since the new Code has such a well-founded antecedent, of some twenty years' standing, it is not surprising that there is little in its content to criticise and much to commend; criticisms may be directed mainly to matters which are omitted from, but which might be expected to be included in, such a Code.

The application of the Code is restricted to structures such as reservoirs and tanks containing water or other aqueous liquids which have no detrimental effect on concrete, and to which concrete is impervious, and which are at normal temperatures. With such limitations, it is thought that the over-long title could well be shortened to, say, "Code of Practice for Concrete Reservoirs."

Since the Code includes prestressed concrete structures, it is good to see the inclusion of the recommendation that, although design should be entrusted only to chartered civil and structural engineers, such engineers should be qualified in reinforced and prestressed concrete. This qualification is most necessary in implementing the requirements of the Code since so many important decisions are left, and quite rightly so, to the engineer to make.

In general, the recommended working stresses in concrete have been increased. In addition to the two mixtures specified in the previous code, namely 1 : 1·6 : 3·2 and 1 : 2 : 4, recommendations for a special mixture of specified strength are given. The tensile stresses recommended in 1 : 1·6 : 3·2 concrete are now 190 lb. per square inch in direct tension and 270 lb. per square inch in bending, compared with 175 lb. and 250 lb. per square inch respectively in the previous code. An explanation of why such small variations were thought to be necessary would be interesting. The compressive stresses in concrete are now in accordance with the recommendations for reinforced concrete in buildings,

* British Standard Code of Practice, CP. 2007 (1960). "Design and Construction of Reinforced and Prestressed Concrete Structure for the Storage of Water and other Aqueous Liquids." Price 12s. 6d. from the British Standards Institution.

but some restriction is quite properly applied to shearing stresses (with or without reinforcement) when designing for resistance to cracking.

The tensile stress recommended in the reinforcement, when designing for strength, is still in general 12,000 lb. per square inch, although in members not less than 9 in. thick the stress in bars near the face remote from the liquid may be 18,000 lb. per square inch in plain bars or 20,000 lb. per square inch in deformed bars. There seems to be no reason why an increased stress should not be allowed in deformed bars under all conditions, not because of their greater strength, but as is admitted in the Code, because of the more diffuse distribution of cracks likely with such bars. The superiority of deformed bars in this respect is also acknowledged in the recommendations for the least amounts of reinforcement required in slabs, which are 0·3 per cent. of the gross cross-sectional area (as before) if plain bars are used, but 0·25 per cent. if deformed bars are used. This requirement is only one of several in this Code which is an improvement on Code No. 114. Another instance is the recommendation of a maximum pitch of 18 in. for bars in a slab.

In other cases, it is not unreasonable to expect more guidance, than is given in the Code, on some arbitrary matters such as those in the following. What smaller percentage of reinforcement is acceptable incerta in floors? How much reinforcement should be provided horizontally at corners of walls designed to span primarily vertically? How should the bending moments on the walls of cylindrical tanks be calculated? (The previous code was explicit on this point.) What protective measures are recommended against corrosion (of what?) due to condensation on the underside of roof slabs? Why must the use of high-alumina cement be outside the scope of the Code in view of its value in combating the effect of corrosive waters? Cannot the "suitability" of aggregates be defined and harmful properties described?

Such omissions cannot be excused on the grounds of lack of space, since the Code is verbose enough on some well-known matters. The phraseology in parts of the Code is not above criticism. Some examples are: ". . . the lining has adequate crack-bridging properties"; ". . . concrete of given mix proportions"; ". . . the water pressure is resisted by both vertical and horizontal bending moments"; "Welding, if used, should be . . ."

The recommendations for joints and prestressed concrete, some of which were in supplements to the previous code, are now incorporated in the body of the new Code, and, in general, the requirements are sound. Some engineers may take exception to the multiplicity of closely-spaced joints as recommended. In reservoirs of medium size, say up to a capacity of a million gallons, it is not uncommon to provide no joints at all so long as the necessary precautions are taken during construction. Such a reservoir, if 12 ft. deep, would be about 115 ft. square, and in accordance with the Code movement joints should be provided in the walls and floor at 20 ft. to 25 ft. centres, and similarly in the roof if it is monolithic with the walls. The precision of these requirements is at variance with the statement that such joints need not be provided if the engineer decides that they are unnecessary. Is the Code addressed to junior designers, who should be told everything in detail, or to engineers who are qualified to decide for themselves?

Design of Slabs by the Yield-line Method.—I.

By A. J. ASHDOWN, A.M.I.Struct.E.

TESTS made by Professor A. J. Ockleston using Johansen's theory of the yield-line method in conjunction with the plastic theory recommended in the British Standard Code of Practice No. 114, show that the calculated failure load of a rectangular slab continuous over four sides and uniformly loaded is about 40 per cent. of the actual load causing failure. It has been suggested by Professor Ockleston that this discrepancy can be explained by arch action of the slab.

Slabs Supporting Uniformly-distributed Load.

Johansen's method of determining the yield moments of resistance of slabs continuous over four edges (*Fig.* 1) is as follows. Let m, m_1, m_2, m_3, m_4, and m_5 be the moments shown in *Fig.* 1 and $i_1 = \frac{m_1}{m}$, $i_2 = \frac{m_2}{m}$, $i_3 = \frac{m_3}{m}$, $i_4 = \frac{m_4}{m_1}$, and $i_5 = \frac{m_5}{m_1}$. Reduced spans given by

$$a_r = \frac{2a}{\sqrt{1 + i_2} + \sqrt{1 + i_3}}, \text{ and } b_r = \frac{2b}{\sqrt{1 + i_4} + \sqrt{1 + i_5}},$$

can be substituted for a and b respectively in Johansen's formula

$$m = \frac{wa^2}{24}\left[\sqrt{3 + \left(\frac{a}{b}\right)^2 i_1} - \frac{a}{b}\sqrt{i_1}\right]^2 \quad . \quad . \quad . \quad (1)$$

A much simpler formula which gives results within about 2 per cent. of those given by formula (1) is obtained by considering the dispersion of the virtual internal work v over the area AC x (*Fig.* 1) to be a minimum. Since $v = \left(4\frac{mx}{a} + \frac{m_1 a}{x}\right)\delta$,

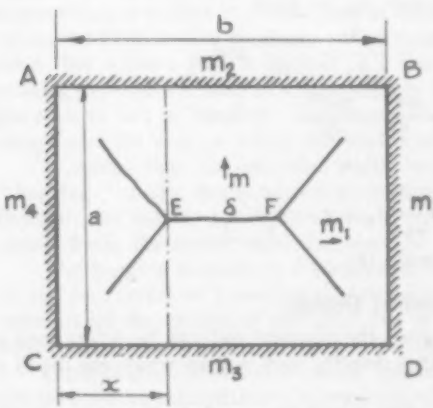


Fig. 1.

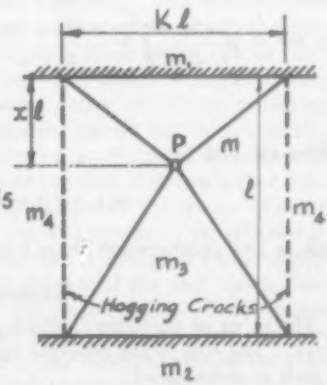


Fig. 2.

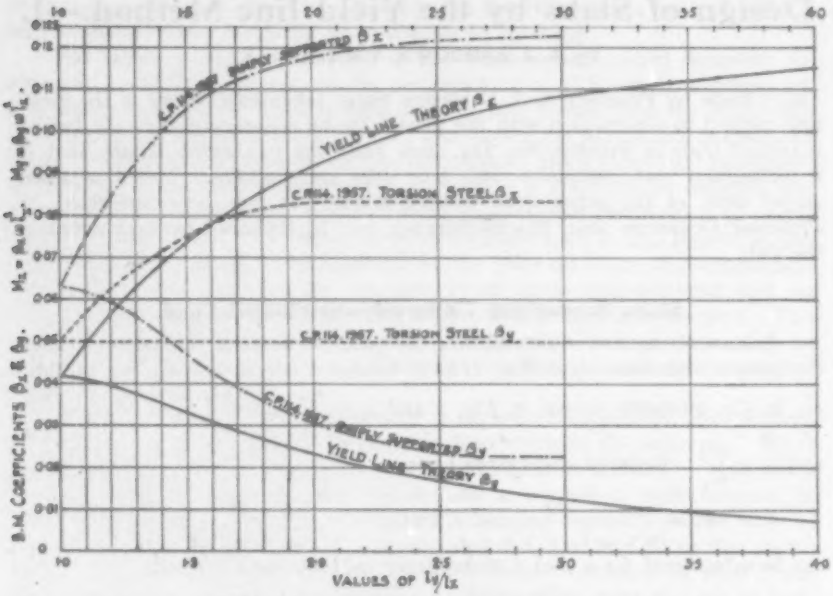


Fig. 3.—Freely-supported Slab; Uniformly-distributed Load.

$\frac{dv}{dx} = 0 = 4\frac{m}{a} - \frac{m_1 a}{x^2}$. Therefore $x^2 = \frac{m_1 a^2}{4m}$, and if $\frac{m_1}{m} = i_1$

$$x = \tfrac{1}{2}a\sqrt{i_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Taking moments in each direction gives

$$mai_1 = \tfrac{1}{8}wax^2;\ mi_1 = \tfrac{1}{8}wx^2 \quad . \quad . \quad . \quad . \quad (3)$$

Also $mb = 2\left(\frac{wxa}{4}\cdot\frac{a}{6}\right) + \frac{wa^2(b-2x)}{2\times 4} = \frac{wa^2b}{8} - \frac{wa^2x}{6}$ and

$$m = \frac{wa^2}{8} - \frac{wa^2x}{6b} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Adding (3) and (4)

$$m(1+i_1) = \frac{wa^2}{24}\left[i_1 + 3 - 2\frac{a}{b}\sqrt{i}\right] \quad . \quad . \quad . \quad . \quad (5)$$

which is less cumbersome than formula (1).

Economical Design.

The value of the ratio i_1 which gives the cheapest slab can be determined as follows. The cost of the concrete varies directly with m from which the depth of the slab is determined.

The quantity of reinforcement is determined by m and m_1, but although the

effective depths to the reinforcement in the two directions differ, the smaller amount of reinforcement in the longer direction and other factors reduce the effect of the difference so that the effective depths can be assumed to be equal.

The cost of the steel s is proportional to $m(1 + i_1)ab$. Substituting in formula (5),

$$s = \frac{wa^2b}{24}\left[i_1 + 3 - 2\frac{a}{b}\sqrt{i_1}\right].$$

For the least value of s, $\frac{ds}{di_1} = 0$, therefore $\frac{wa^2b}{24} - \frac{wa^4}{24\sqrt{i_1}} = 0$, from which $i_1 = \left(\frac{a}{b}\right)^2 = K^2$. From formula (5),

$$m = \frac{wa^2}{24}\left(\frac{3 - K}{1 + K^2}\right) \quad . \quad . \quad . \quad . \quad . \quad (6)$$

In designing a slab, m is found from formula (6); then $m_1 = i_1 m = K^2 m$. *Fig.* 3 gives values of m calculated from formula (6) compared with coefficients given in B.S. Code No. 114 and indicates clearly the saving which results from using the yield-line method.

Since torsion at the corners of the slab is not considered in the analysis for uniformly-distributed load, extra reinforcement to resist torsion appears to be unnecessary, unless the slab supports a heavy concentrated load.

If the reinforcing bars provided at midspan are bent up into the top of the slab over both supports, the reduced spans, $a_r = 0{\cdot}707a$ and $b_r = 0{\cdot}707b$, can be substituted for a and K in formula (6). If the bars are bent up over only one support, $a_r = 0{\cdot}83a$ and $b_r = 0{\cdot}83b$.

Effective Width of Slab Supporting a Concentrated Load.

Tests made by Professor A. J. Ockleston indicate that, under concentrated loads or loads on narrow strips, slabs with large ratios of width to span have larger effective widths at failure than is usually assumed. The effective width is influenced little by the thickness or by the proportion of longitudinal reinforcement, but when a load is applied on a small area there is the risk of the load pushing through the slab at a load slightly less than would be required to cause the slab to fail in bending. Resistance to this action should be considered in design and the area on which the load is imposed should not be too small.

It seems that the effective width increases as the load increases before " hogging " cracks, due to tension at the top of the slab, form on either side of a concentrated load or a load on a narrow strip parallel to the span. When these cracks form, the entire width between the cracks appears to be effective.

The hogging moment of resistance of the slab is increased considerably due to the production of a compressive stress in the top of the slab by the bending moment at the middle of the span due to the dead weight. The compressive stress has to be relieved before tension and subsequent cracking can occur.

Shrinking of the concrete causes tension in the concrete around the bars and tends to reduce the resistance to cracking, which therefore depends on the amount of reinforcement. Allowance for the foregoing states of stress should be made

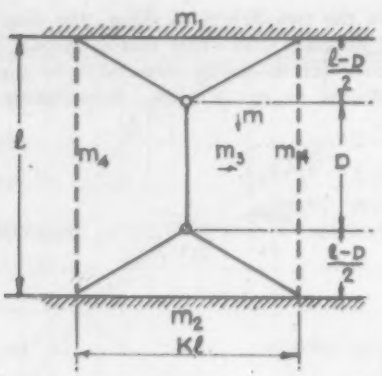


Fig. 4.

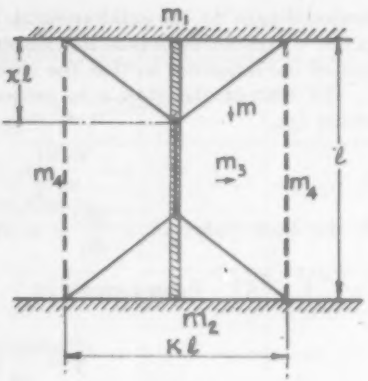


Fig. 5.

when postulating relative moments of resistance to cracking in calculating the effective width.

For the purpose of the following analysis, it is assumed that cracking occurs in accordance with the yield-line theory. After cracking has occurred the slab is assumed to act as a simple slab with an effective width equal to the distance between the hogging cracks (*Fig.* 2).

To determine the position of the hogging cracks it is assumed that, at cracking, the total virtual internal work is a minimum, so that the work in the transverse direction is equal to the work in the direction of the span.

Let m be the mid-span moment, in the direction of the span, which causes cracking, m_1 and m_2 the moments, over the opposite supports, which cause cracking, m_3 the mid-span moment in the transverse direction, and m_4 the hogging moment (*Fig.* 2). Then for a load concentrated at a distance xl_1 from the support the virtual internal work is as follows.

In the direction of the span: $(m + m_1)\dfrac{Kl}{xl} + \dfrac{(m + m_2)Kl}{(1 - x)l}$.

Transversely: $2(m_3 + m_4)\dfrac{2l}{Kl}$.

Total work: $V = (m + m_2)\dfrac{K}{x} + \dfrac{(m + m_2)K}{1 - x} + \dfrac{4(m_3 + m_4)}{K}$.

$$\frac{dV}{dK} = 0 = \frac{(m + m_1)}{x} + \frac{(m + m_2)}{1 - x} - \frac{4(m_3 + m_4)}{K^2}.$$

If $$m_1 = m_2 = m,\ K^2 = \frac{2(m_3 + m_4)}{m}x(1 - x) \quad . \quad . \quad . \quad (7)$$

Assuming that the cracking moment $m_2 = 1{\cdot}33m$, and $m_4 = 1{\cdot}1m$; $K^2 = 4{\cdot}86x(1 - x)$. If $x = 0{\cdot}5$, $K = 1{\cdot}10$. If $x = 0{\cdot}25$, $K = 0{\cdot}96$. Let the yield moments of resistance per unit width be M in the span, M_1 over the nearest support, and M_2 over the further support. Then

$$M + M_1(1 - x) + M_2x = \frac{P(1 - x)x}{K} \quad . \quad . \quad . \quad (8)$$

from which the load causing failure can be found, provided that the load does not push through the slab. The width of the loaded area is neglected in this analysis.

Two Concentrated Loads Symmetrically Disposed.

In the case of two concentrated loads a distance D apart and acting symmetrically on a span (*Fig.* 4),

$$(m + m_1)\frac{2Kl}{l - D} + (m + m_2)\frac{2Kl}{l - D} = 2(m_3 + m_4)\frac{2l}{Kl},$$

from which
$$K^2 = \frac{2(m_3 + m_4)(l - D)}{(2m + m_1 + m_2)l} \quad . \quad . \quad . \quad . \quad (9)$$

If $m = m_1 = m_2$, and $m_3 = 1{\cdot}33m$, $m_4 = 1{\cdot}1m$, and $D = \frac{1}{2}l$, then $K = 0{\cdot}78$ and

$$M + \frac{M_1 + M_2}{2} = \frac{P(l - D)}{2Kl} \text{ (approximately)} \quad . \quad . \quad . \quad (10)$$

Load Concentrated and Strip Parallel to the Span.

In the case of a load concentrated on a strip (*Fig.* 5), the points at which the diagonal cracks radiate from the load have to be determined first. The requirement is that the total virtual work is a minimum, and that the value of K satisfies the equation for equal work in two directions.

Virtual external work $U = P[2(\frac{1}{2}x) + 1 - 2x] = P[1 - x]$.

Virtual internal work $V = (m + m_1)\frac{K}{x} + (m + m_2)\frac{K}{x} + (m_3 + m_4)\frac{4}{x}$;

to simplify this equation assume that $m_1 = m_2$, then

$$V = 2(m + m_1)\frac{K}{x} + \frac{4(m_3 + m_4)}{K}.$$

Since $U = V$, $P = \frac{2(m_1 + m)K}{x(1 - x)} + \frac{4(m_3 + m_4)}{K(1 - x)}$; then if K is constant, for the minimum value of P, $\frac{dP}{dx} = 0 = \frac{-2(m_1 + m)K(1 - 2x)}{[x(1 - x)]^2} + \frac{4(m_3 + m_4)}{K(1 - x)^2}$,

from which $-2(m_1 + m)K^2(1 - 2x) + 4x^2(m_3 + m_4) = 0 \quad . \quad . \quad . \quad (a)$

Equating the work in two directions, $K^2 = \frac{2(m_3 + m_4)x}{m + m_1}$; substituting in expression (a), $x - (1 - 2x) = 0$; hence $x = \frac{1}{3}$.

If $m_3 = 1{\cdot}33$ and $m_4 = 1{\cdot}1$, $K^2 = \frac{2}{3} \times \frac{2{\cdot}43}{2}$ and $K = 0{\cdot}9$. If $m_1 = 0$, $K = 1{\cdot}27$.

Let the yield moment of resistance per unit width be M in the span, M_1 over one support, and M_2 over the opposite support. Then approximately

$$M + \frac{M_1 + M_2}{2} = \frac{P}{8K} \quad . \quad . \quad . \quad . \quad . \quad (11)$$

(To be continued.)

Book Reviews.

"Concrete Formwork Designer's Handbook." By H. R. Gill. (London: Concrete Publications, Ltd. 1960. 160 pages. Price 15*s.*; by post 16*s.*; in Canada and U.S.A. 3.50 dollars.)

THIS book is probably unique in that it gives in a concise form all the practical data required for the design of the formwork, or shuttering, for walls, slabs, beams, and the like. The several factors which affect the pressure of newly-placed concrete against the formwork are evaluated, and the stresses permissible under various conditions in timber, plywood, and steel are given. Formulæ for the calculation of the bending moments and forces on sheeting, walings, and ties for forms for vertical and inclined walls, and on beam-and-slab shuttering are expressed in a manner sufficiently accurate for design purposes; the derivation of the formulæ are given in an appendix, thereby freeing the practical part of the book from intrusions of theory.

The feature of the book is the aids to rapid design which are in the form of numerous nomograms, supplemented by tables and charts, from which the sizes of various components can be determined in the case of vertical and horizontal forms, studs and soldiers, joists and bearers, walings, ties, and scaffold tubes. The application of the design data is explained clearly by means of several fully-worked examples. The shuttering of curved walls and dams, and continuously-moving forms are also dealt with. This book is a useful addition to the several other books in this series dealing with shuttering and moulds for concrete.

"Theory of Plates and Shells." By S. Timoshenko and S. Woinowsky-Krieger. (London: McGraw-Hill Book Co., Ltd. Second edition. 1959. Price 116*s.* 6*d.*)

THE principal additions to this well-known mathematical treatise on thin structural members such as flat and curved plates, in which are included reinforced concrete slabs and "shells", include consideration of plates (or slabs) on elastic foundations, the bending of anisotropic (non-isotropic) plates, and a review of some special and approximate analyses of plates. Other additions include the deflection of plates due to transverse shearing, stress concentrations around circular holes in bent plates, and extension of the chapter on plates subjected to large deflections. The chapter dealing with continuous rectangular plates may be of most interest to designers of reinforced concrete structures since, although the theoretical analysis is abstruse, the application of the theory is made easy by the provision of tables for panels continuous over one or more edges. The theory of flat-slabs is also dealt with, but again the mathematical treatment is more suitable for students of elasticity than for practising engineers.

As regards "shell" roofs, the text is much the same as in the previous edition except for the addition of a stress-function method of dealing with the membrane theory, and the theory of shallow domes. The authors mention recent advances in the methods of designing "shells", a bibliography of which is given.

"Mehrfeldrahmen." Vol. I. By A. Kleinlogel and A. Haselbach. (Berlin: Wilhelm Ernst & Sohn. 7th edition. 1959. Price 68 D.M.)

FORMULÆ are given for the bending moment and forces on seventy examples of multiple-span rectangular frames of one story, and twenty examples of similar frames of two stories. The frames are assumed to be fixed at the supports, and side sway is taken into account. Frames having up to eight spans are considered. For each example a general solution is provided from which particular solutions can be obtained by means of a matrix of a quadratic type. The calculations are based upon consideration of forces and deformations, thereby giving the designer a choice of methods. The solution for horizontal displacements is derived from the case of fixed supports with the aid of an additional equation of elasticity. The intermediate stages of the calculation are clearly indicated. The authors claim that their methods are superior to moment-distribution methods for multiple-span frames if several loads have to be considered.

Frames with sloping members, loads applied at an angle, and the effects of changes of temperature are dealt with in an appendix.

The Kariba Dam.

Fig. 1.

THE Kariba Hydro-electric Scheme for the Federal Power Board acting as agents for the Government of the Federation of Rhodesia and Nyasaland was officially inaugurated last month by He: Majesty Queen Elizabeth the Queen Mother. The scheme includes a dam (*Fig.* 1) across the River Zambesi, 180 miles north-west of Salisbury. The dam is a plain concrete arch structure having a maximum height of 420 ft. and is 2025 ft. long at the top where there is a road. The greatest thickness is 80 ft.

Two power-houses are included in the scheme, the first, which is already in operation, having a capacity of 600,000 kW. The second, construction of which is to start in 1962, will have a capacity of at least 900,000 kW. The present power-station (*Fig.* 3) is underground in the southern bank. Each turbine has a separate intake and penstock and the intakes, which are in the side of the gorge, have screens which can be cleaned from a gantry above water-level. The permanent intake tunnels, temporary intake tunnels, and the outlet tunnels serving the power-station are shown in *Fig.* 3.

Design.

A cross-section of the dam is shown in *Fig.* 4. The design is based on observations obtained from tests on plaster and kieselguhr models. Loads simulating those to which the main dam will be subjected were applied to the models by means of several small hydraulic jacks or by thrusts from a rubber bag filled with mercury. The resulting strains were measured by electrical strain-gauges attached to each face of the model, and by deflection gauges having an accuracy of 0·01 mm. Calculations were then carried out with the aid of a computor to confirm the design.

Construction.

The dam was constructed in four phases. In the first phase a semicircular cofferdam (*Fig.* 2) was constructed on the northern bank of the river. The water was pumped out of the cofferdam and the first part of the main dam, through which temporary openings were formed, was constructed inside the cofferdam to a height of 60 ft. A diversion tunnel was excavated through

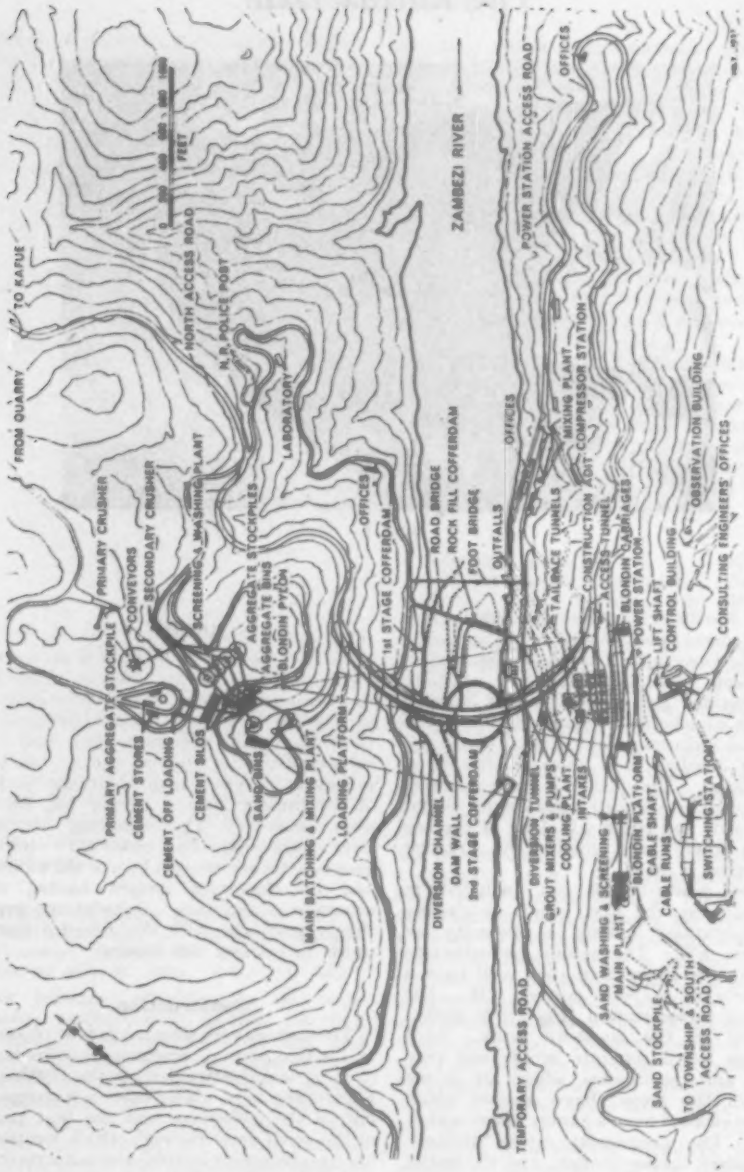


Fig. 2.—General Arrangement of Works.

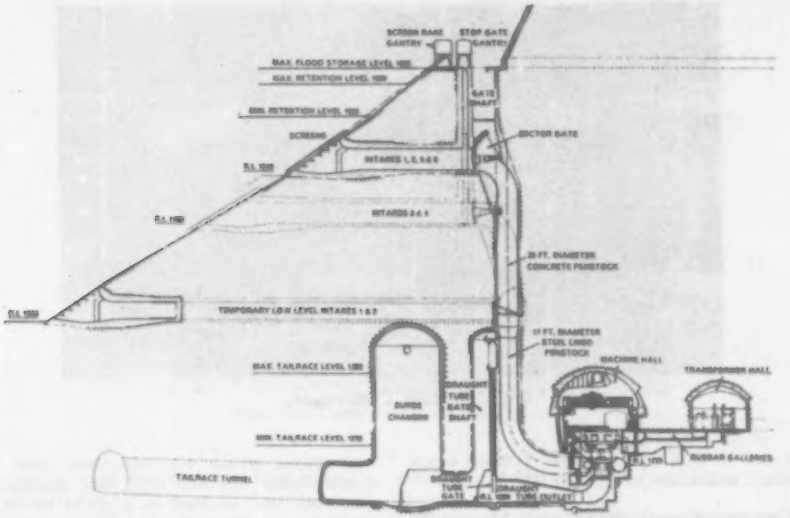


Fig. 3.—Cross-section of Power-house and Tunnels.

the southern bank and a diversion channel dug upstream and downstream from the temporary openings.

In the second phase the river was diverted through the diversion tunnel and channel by rock filling placed from a temporary road bridge (*Fig.* 2). Gaps were blasted in the cofferdam at the northern bank to allow the river to flow along the diversion channel and through the temporary openings in the main dam (*Fig.* 7). The main circular cofferdam was then built between two rings of steel sheet-piling.

The third phase comprised the construction of the central part of the dam within the circular cofferdam (*Fig.* 5) and the continuation of the construction at the northern bank. During this phase the river rose to an unprecedented height and the main cofferdam was flooded and the road bridge swept away. The resulting delay was overcome by replanning and accelerating later work.

In the fourth phase the diversion tunnel in the southern bank and the temporary openings in the dam were blocked, at which time the dam was 250 ft. high. The dam was completed while the water was being impounded. The dam was constructed in blocks about 50 ft. wide and 7 ft. 6 in. high. The radial joints between the blocks were subsequently grouted under pressure.

The concrete was cooled while setting by chilled water passing through pipes embedded in the concrete. Holes were drilled and grout was injected to form

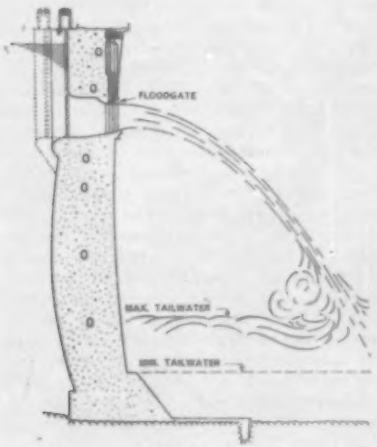


Fig. 4.—Cross-section of Dam.

Fig. 5.—Main Cofferdam.

a cut-off wall 80 ft. to 160 ft. deep below the foundation.

Constructional Plant and Materials.

The coarse aggregate for the concrete was gneiss which was obtained from a quarry near the site and was crushed, screened, and washed in a plant on the northern bank (*Fig.* 8). The material was graded and stored in four sizes: from 8 in. to 3 in., 3 in. to 1 in., 1 in. to ¼ in., and ¼ in. to $\frac{1}{16}$ in. The fine aggregate was obtained from deposits of river sand which was screened and classified into four sizes by a dynamic sedimentation process at a plant on the southern bank (*Fig.* 2). Because the reservoir will cover these deposits, sufficient sand for the concrete required for the second-stage power-station has been stored in stock-piles. The monthly capacity of the works supplying ordinary Portland cement was smaller than the greatest monthly requirements of the dam. The necessary stocks were therefore provided by storing about 21,500 tons of cement in eighty silos and about 3600 tons in sheds.

Fig. 6.—Mixing and Batching Plant.

The batching and mixing plant (*Fig.* 6) was designed to produce 360 cu. yd. of concrete in an hour. The actual production was up to 100,000 cu. yd. per month. The concrete generally had a water-cement ratio of 0·5 to 0·6 and contains 330 lb. of cement per cubic yard. For a thickness of 10 ft. on the face of the dam in contact with water, the concrete contains 430 lb. of cement per cubic yard.

Concrete was placed in the dam by means of 8-cu. yd. buckets suspended from three cableways and was compacted by means of immersion vibrators. The

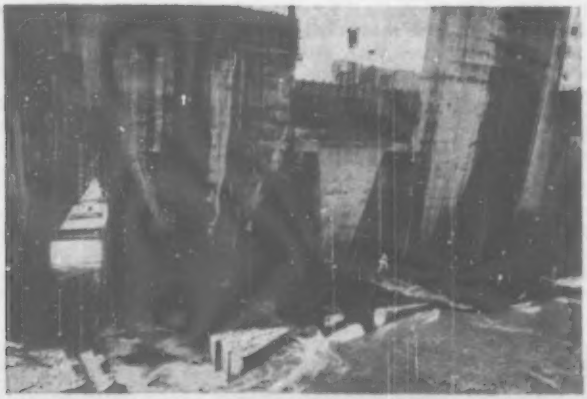


Fig. 7.—Temporary Openings.

shuttering was described in this journal for March 1957. Among the preliminary works for the project were two access roads, a road bridge, a suspended foot-bridge, a cable-way, main and secondary mixing plants, and a refrigeration plant with a capacity of 10,500,000 B.t.u. per hour.

The consulting engineers are Messrs. Gibb, Coyne, Sogei (Kariba) (Pvt.), Ltd., a consortium formed by Sir Alexander Gibb & Partners, Messrs. A. Coyne

Fig. 8.—Concrete Materials Plant.

& J. Bellier, and Société Générale d'Exploitations Industrielles. The main civil engineering work in connection with the construction of the dam and the underground power-house will cost about £25,300,000 and is being carried out by Impresit (Kariba) (Pvt.), Ltd.; British firms carried out most of the preliminary work. Cementation Ltd., and Messrs. John Laing & Son (Rhodesia), Ltd., built the cofferdam on the northern bank, the diversion tunnel, channel and other works at a total cost of £2,200,000. Messrs. Richard Costain (Southern Rhodesia), Ltd., constructed the houses and associated works.

An Architectural Award.

THE London Architecture Bronze Medal, which is awarded annually by the Royal Institute of British Architects for a building of exceptional merit completed during the preceding three years, was presented this year to the London County Council for the group of high blocks of residential flats at Roehampton. The structures, which are of composite precast and cast-insitu reinforced concrete construction, and some of which are shown in the accompanying illustration, were designed by the Architect's Department of the Council, the head of which at present is Mr. Hubert Bennett; the housing scheme, of which the flats form a part, was begun while Professor Robert Matthew was architect to the Council. The general contractors were Messrs. Wates Ltd.

Bridge Girders Prestressed with Large Strands.

A TWO-STORY office building constructed over the Grand Union canal at Southall, Middlesex, is supported by two concrete girders, having a clear span of 79 ft. and each precast in eight parts about 12 ft. long, and a cantilever of 15 ft. prestressed together with steel strands of a nominal diameter of 1¼ in. The parts of the girders are reinforced and were precast at a factory, and transported about seven miles by road to the site. In making these parts the reinforcement was assembled on a steel pallet and the sheaths of 1¼ in. diameter which formed the ducts for the strands were supported in position by formers (*Fig.* 3). The straight sheaths at the bottom were stiffened by scaffold tubes and the curved sheaths were stiffened by inflated rubber cores. When the steel sides and end-plates of the mould were in position, concrete made with crushed Cornish granite, sand, and white cement was placed in the mould and compacted by means of vibrators attached to the sides of the mould and by immersion vibrators (*Fig.* 4). The concrete had a crushing strength at 28 days of at least 9000 lb. per square inch.

At the construction site the parts of the girder were assembled on scaffolding on a nearby road bridge (*Fig.* 1) with a gap of ¾ in. between adjacent parts. Rubber tubes were passed into the ducts and inflated, and shuttering was placed at the sides and bottoms of the joints. Mortar made with white cement and sand in the proportions of 1 : 1½ and with a water-cement ratio of 0·4 was pumped into each joint through holes in the bottom shutter by means of a pump developed by the contractors. When the mortar had hardened the tubes were withdrawn and strands were drawn through the ducts by means of a hand-winch. Forces of up to 2 tons were applied to each strand to overcome friction. The strand contains nineteen wires of Warrington lay (6 and 6 over 6 over 1) and has a breaking strength of at least 181,000 lb. The losses during and after prestressing were calculated to be as follows: Due to gradual application of the prestress, 4170 lb. per square inch; due to shrinking of the concrete, 4600 lb. per square inch; due to creep of the concrete, 10,400 lb. per square inch; due to relaxation of the strand,

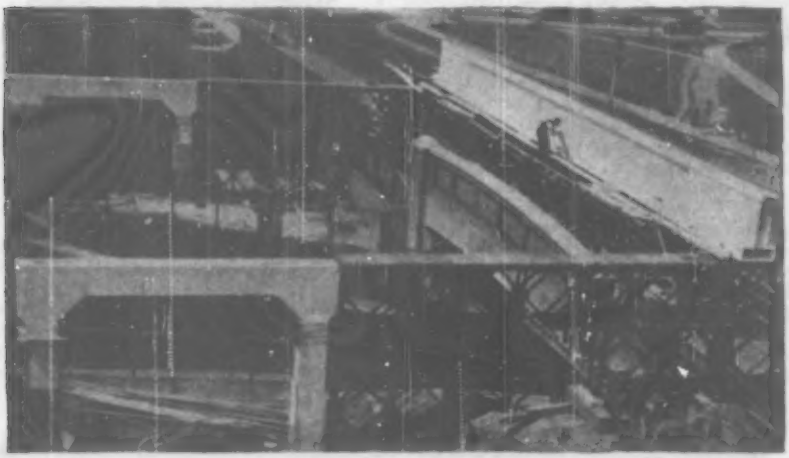


Fig. 1.—Construction in Progress.

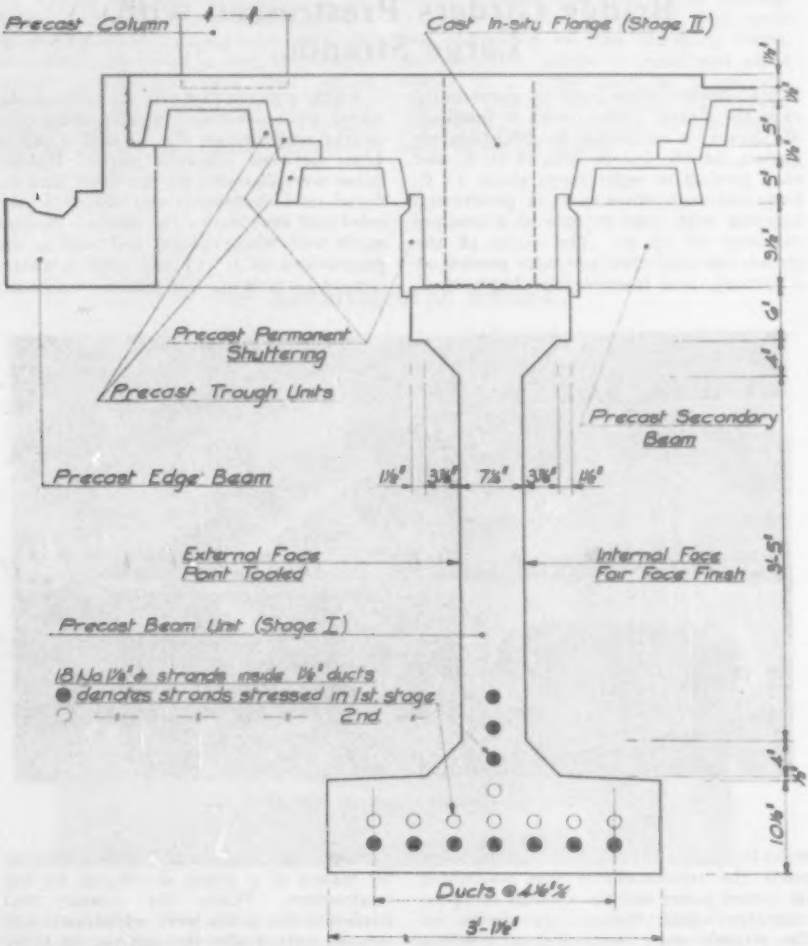


Fig. 2.—Typical Section of Beams.

7600 lb. per strand. The sum of these losses is 23,600 lb. per strand. The design load is 103,100 lb. per strand and the initial force applied to each strand was therefore 126,700 lb.

The prestressing was carried out in two stages; in the first stage ten strands were tensioned to enable the beam to support its own weight and that of the deck, and in the second stage the eight remaining strands (*Fig.* 2) were tensioned. The strands were tensioned at one end and each is anchored between three steel wedges drawn into a large steel cylinder which thrusts on a bearing-plate embedded in the member; one of the anchors is shown on the topmost strand in *Fig.* 5.

The recommended distance between adjacent strands is at least 6 in., but in this case, by the use of more reinforcement

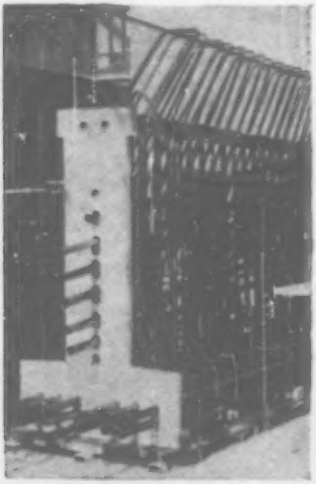


Fig. 3.—Reinforcement and Sheaths for Prestressing Cables.

near the anchorages, it was possible to place them at 4½ in. centres.

The strands were tensioned by means of a large oil-operated jack (*Fig.* 5). A wedge-anchor was first threaded on to the strand. The jack, supported by a chain hoist, was then threaded on and a temporary anchor fixed to the strand behind the jack. The jack is powered by oil-pressure from a motor-operated pump and is capable of three actions, namely (1) extension of the main ram thrusts the body of the jack backwards and extends the strand; (2) when the required extension is reached a secondary ram thrusts the wedges into the anchor; (3) the primary ram is retracted and the temporary anchor and jack can be removed.

In this case an initial tension of 5 tons was applied to remove bends and take up slackness. Tension was then applied in increments of 10 tons up to 60 tons and the strand was anchored. After a few minutes the tension had reduced to 58½ tons and the final tension was expected to be about 57½ tons.

Data obtainable during the tensioning of a typical strand 97 ft. long are as follows. It was assumed that when a force of 5 tons was applied the extension of the ram was zero; the extensions were ½ in. at 10 tons, 1 1/16 in. at 20 tons, 2⅛ in. at 30 tons, 4 1/16 in. at 40 tons, 5 9/16 in. at 50 tons, 6½ in. at 55 tons, and 7 in. at 57½ tons when the pressure-gauge attached to the pump indicated 3400 lb. per square inch. After two minutes the residual tension was 57 tons. The wedges slipped a total distance of 9/16 in.

The force applied to the strand was measured by the electric meter supplied

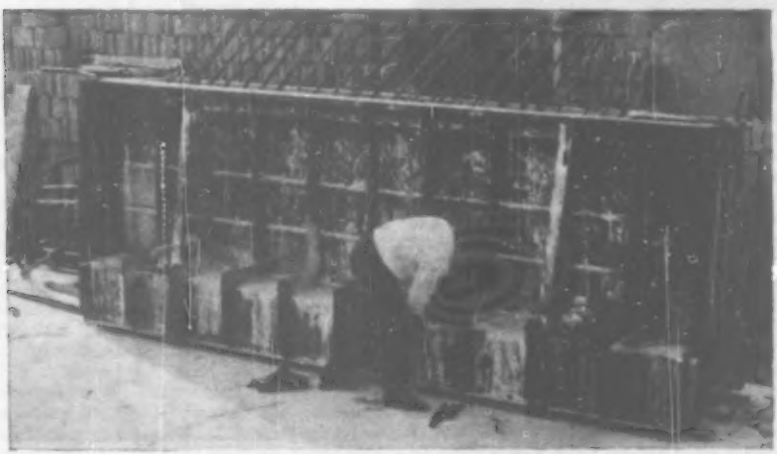


Fig. 4.—Steel Mould and Vibrators.

Fig. 5.—Tensioning the Cables.

with the equipment and placed on the lower flange of the girder (*Fig.* 5). The measuring equipment comprises a load cell which is incorporated in the jack, a pressure gauge, and a meter which gives audible and visual indication of the force applied to the cell. The cell consists of a metal cylinder through which the force applied by the jack is transmitted to the strand. As this cylinder is strained, gauges attached to the side measure the strain and transmit information to the meter, which is a modified Wheatstone bridge.

For these girders the meter was first adjusted so that the pointer on the dial was as nearly as possible at the centre of the scale when no force was applied to the load cell. The large rotating indicator

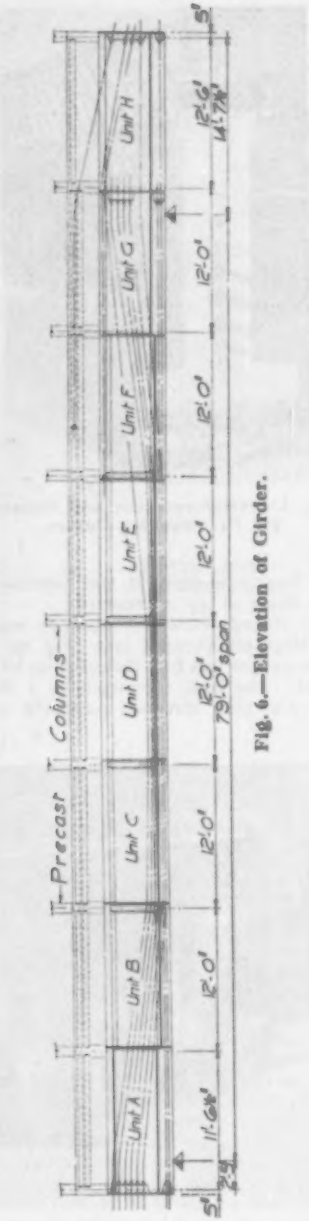


Fig. 6.—Elevation of Girder.

on the meter was set at 5 tons, which was the first tension to be applied. The pointer on the dial moved and a whistle became audible in the earphone. The jack was then extended until the pointer returned to its original position, when the sound of the whistle was faintest, thereby indicating that a force of 5 tons had been applied to the strand. The reading on the indicator was increased by 10 tons and the jack operated as before until the tension in the strand was increased by 10 tons. This procedure was repeated until a force of 60 tons had been applied to the cable, when the anchoring wedges were driven into the cylinder and the jack was removed. The extension of the jack and the amounts that the wedges had been drawn into the temporary anchorage at the rear of the jack and into the permanent anchorage at the other end of the beam were noted after each increase of tension.

At one end of the bridge the girders cantilever about 15 ft. beyond the support and extend to within 1 in. of the wall of an adjoining building. The second-stage anchorages at these ends and some which are contained in the last joint between precast parts (*Fig.* 6) are not accessible and had to be encased in grout before the prestress was applied to the beams. The cylinders of these anchorages were covered with cardboard sheets before grouting the joints, so that the cylinders could expand up to 0·01 in. under the action of the wedges without damaging the grout or concrete.

Erecting the Beams.

When all the strands in the first stage had been tensioned each girder was moved to its final position along elevated runways about 40 ft. long (*Fig.* 1). The runways, which were supported by Bailey-bridge panels, consisted of baulks of timber on which a layer of grout was placed. A strip of steel about 2 ft. wide and $\frac{1}{16}$ in. thick was embedded in the grout to provide a smooth surface. Each girder was moved on four small trolleys at each end of the girder (*Fig.* 7); each of the trolleys was of 10 tons capacity.

Fig. 7.—The Trolleys.

Hand-winches and single sheaves (*Fig.* 7) were used to draw the beams along the runways and were fixed on small towers beside the final supports. The greatest force exerted during this operation was about 3 tons. Winches were also placed on the road bridge to control the movement of the beams.

Oil-operated jacks were used to raise the ends of the beams from the trolleys and, when the trolleys were removed, to lower the beams on to their seating. The jacks were placed under the beam in line with the web, and shims placed under the lower flange to keep the beam balanced were removed as the beam was lowered. The strands were grouted by means of a grout pump, the trapped air escaping from the duct through small copper tubes cast in the concrete.

When the main girders were in position precast transverse beams and precast columns were placed in position, and the

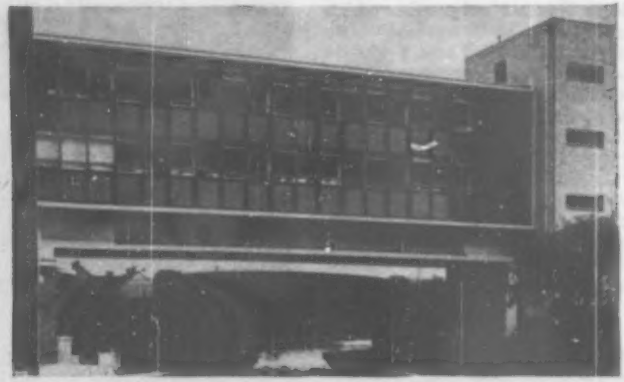


Fig. 8.

floor slab of the building and the top flange of the girders were cast in place on precast concrete permanent shuttering. The second stage of prestressing was then carried out and the remainder of the building completed. All the visible concrete is made with white cement and the webs of the main girders have been bush-hammered to expose the aggregate. An illustration of the completed bridge is given in *Fig.* 8.

The general contractors are Messrs. Taylor Woodrow Construction, Ltd. The sub-contractors for the design and construction of the prestressed and precast concrete were Pierhead, Ltd.

Bridge Girders with Long Reinforcement Bars.

A FEATURE of the widened bridge which carries Watford Way, London, over Bunns Lane and the adjacent railway is the use of reinforcement bars up to 90 ft. in length. The total length of the structure is 150 ft. but virtually there are two bridges, one over the railway (at present a single line) and one of longer span over the road. The width of the widened part, which comprises four longitudinal girders compared with ten in the original structure, is 22 ft. The deck is of monolithic beam-and-slab construction. The abutments and wing walls are also of reinforced concrete. The bars 90 ft. long are in the girders over the road (*Fig.* 1), which are 4 ft. 6 in. deep and act as freely-supported beams.

Fig. 1.

The widening was carried out while maintaining traffic on Watford Way, Bunns Lane and the railway. The bridge, which cost £50,000, was designed and erected under the supervision of Mr. H. S. Andrews, M.I.C.E., Engineer of the Middlesex County Council. The contractors were Messrs. Howard Farrow, Ltd.

A Large Prestressed Water Tank.

SLIDING SHUTTERS.

A PRESTRESSED concrete water tank (*Fig.* 1) with a capacity of 2,000,000 gallons recently built at Tyler, Texas, U.S.A., is believed to be the largest of this type in existence. The internal diameter is 98 ft. 6 in. and the depth 35 ft. ; the height of the substructure is 160 ft.

The substructure comprises three concentric concrete rings, the inner two being plain cylinders and the outer one fluted. The diameters of the inner rings are 37 ft. 6 in. and 70 ft. 4 in. and the least diameter of the fluted ring is 103 ft. 4 in. All the rings are 8 in. thick. The foundation slab is of reinforced concrete; it is 4 ft. 3 in. thick, and comprises 1300 cu. yd. of concrete (containing 470 lb. of cement per cubic yard) and 44 tons of reinforcement. The concrete was placed in one operation during a period of twenty-seven hours.

The sliding shutters used for the construction of the walls were supported by yokes, each secured to an hydraulic jack mounted on a 1-in. rod projecting vertically from the placed concrete (*Fig.* 2). The yokes resisted the lateral pressure exerted on the shutters by the concrete, and the force produced by the jacks was applied to the shutters by means of attachments on the yokes which fitted under the lower walings of the shutters. Each jack contained two sets of jaws ; when the hydraulic pressure was applied the lower jaws grasped the rod and the shutters were raised, and when the hydraulic pressure was released the upper jaws grasped the rod while the lower jaws were raised by means of a spring in readiness for another lift. The jacks were connected to one pumping plant, thereby ensuring uniform movement of the shutters.

Three pumps, one of which was a stand-by, and 100 yokes and jacks were used ; each pump was driven by a 2-h.p. electric motor. The jacks were raised 1 in. at a time and the average rise per hour was 9 in. As the work progressed additional sections were added to the jack rods, the ends of which were tapped so that they could be connected by ¾-in. studs. The rods were surrounded by tubes suspended from each jack, which formed holes in the concrete around the rods. When the work was completed the rods were recovered.

Fig. 1.

The shuttering comprised tongued-and-grooved timber, 4 in. by 1 in. in cross section and 45 in. long, and walings of 6-in. by 2-in. timber shaped to the correct radius. Girders made of reinforcing bars were placed between the concentric shutters to act as supports for the working platforms and the shuttering for the floor of the tank.

The reinforcement of the walls consists of ½-in. bars placed horizontally and vertically at 12-in. centres at the centre of each wall. The vertical bars were held in place by jigs attached to the shutters, and the horizontal bars were fixed to the vertical bars as the work proceeded.

The concrete was placed continuously, except for an interval of one day, after the shutters had been raised about 3 ft. 6 in., to enable scaffolding to be suspended from the outer ring. This provided a platform from which the concrete was finished with wooden floats.

The concrete was mixed on the ground, emptied into a bucket with a capacity of 1 cu. yd., and hoisted to hoppers on the

working platform, whence it was transported to the shutters in barrows.

After the working platform had reached its final position, in which it was to support the shuttering for the bottom slab of the tank, ⅝-in. bars 18 in. long and pointed at one end were driven through the shutters and into the concrete immediately below the upper walings at centres of 2 in.; these transmitted the weight of the slab and shutters to the walls. The pumps, yokes, and oil-pipes were then dismantled, the jack-rods were recovered, and the platform removed.

The substructure was completed nine days after the one-day interruption, 3010 cu. yd. of concrete containing 610 lb. of cement per cubic yard having been placed in 262 hours. Two shifts, each of twelve hours, were worked; one foreman and thirty men were employed on each shift.

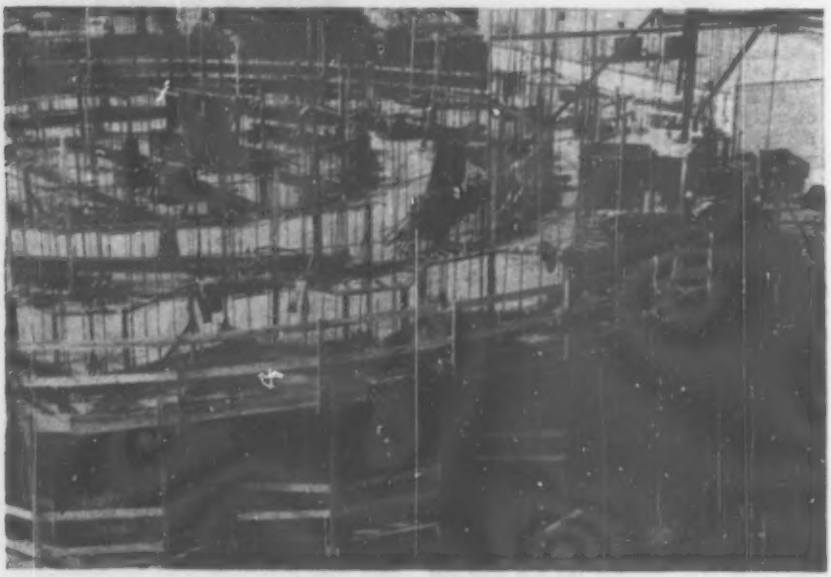


Fig. 2.—Erection of Sliding Shutters.

The Tank.

The bottom of the tank is 2 ft. thick and comprises 738 cu. yd. of concrete (containing 610 lb. of cement per cubic yard) and 38 tons of reinforcement. The concrete was placed continuously during 48 hours. A continuous rubber water-stop was cast into the floor in a position corresponding to the centre-line of the wall of the tank, and ⅝-in. by 1¼-in. rubber pads were cemented to the floor on each side of the water-stop. The areas between the rubber pads and the edges of the wall were covered with removable strips of rubber of the same thickness as the pads. This method of construction enables the walls and floor of the tank to deform without causing secondary stresses at the junction.

The central access shaft and the wall were constructed with sliding shutters, and 78 yokes and jacks were used. Working platforms and guard-rails were provided where required. The walls of the tank have the same amount of mild steel reinforcement as the supporting walls. Vertical prestressing bars, ¾ in. in diameter, were also built into the wall at centres of 4 ft. They were encased in metal sleeves and fitted with anchor-plates at the top and bottom of the wall.

After the prestressing force had been applied the bars were grouted. A rubber pad ⅜ in. thick was cemented to the top of the wall (*Fig.* 3).

The shuttering for the domed roof comprised 6-in. by 1-in. timbers which were supported on girders made of reinforcing bars spanning between temporary columns and the wall. The reinforcement comprises 9100 sq. ft. of wire mesh and 3½ tons of bars. The thickness of the dome varies from 8 in. at the edge to 4 in. at the centre ; there are no internal supports. The roof was cast in four parts.

After the wall and roof had been concreted they were prestressed by winding around the wall about 10½ tons of high-tensile wire, drawn to a diameter of 0·141 in. from the original diameter of 0·162 in.

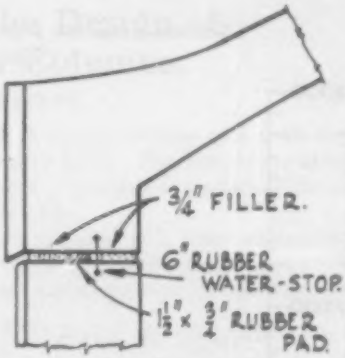


Fig. 3.—Joint between Roof and Wall.

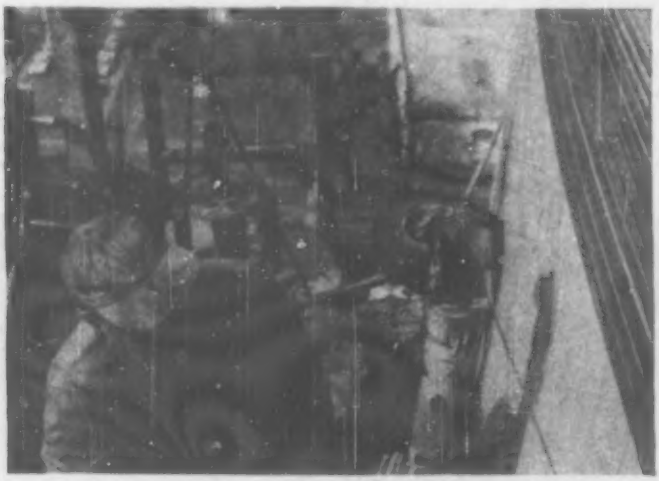
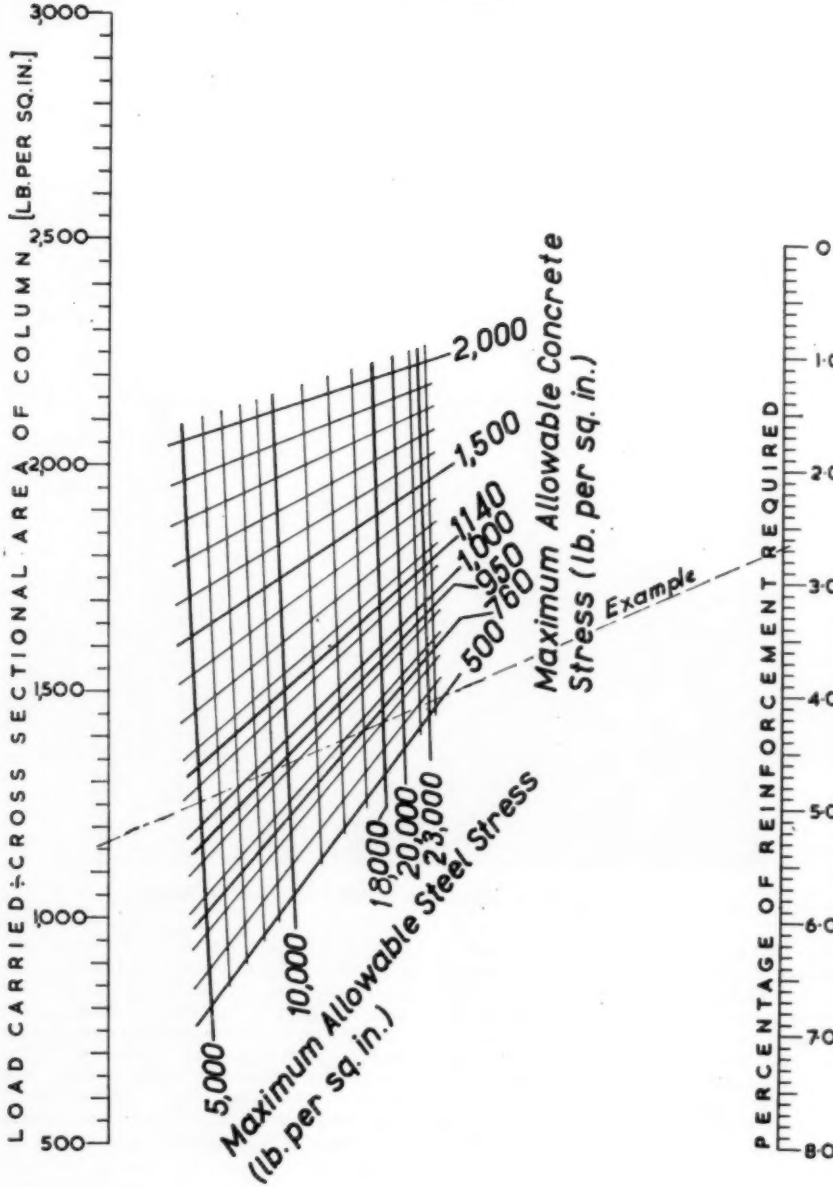


Fig. 4.—Tensioning Circumferential Prestressing Steel.

The initial prestressing force varied from 140,000 lb. per foot at the base of the wall to 17,300 lb. per foot at the top. The prestressing force applied to the dome was 245,000 lb. The rubber pads on which the wall and dome are supported deformed as the prestress was applied, thus preventing the development of secondary bending moments. A layer of mortar about 1 in. thick was applied by a cement gun over the exterior of the tank and the edge of the dome to protect the wire.

The wire was tensioned by means of a machine (*Fig.* 4) suspended from a carriage which travelled along the outer edge of the dome and was connected to the centre of the dome by radial cables ; the entire mechanism was self-propelled. The tank was designed by the Preload Co., Inc., of New York.

NOMOGRAM FOR THE DESIGN OF AXIALLY-LOADED COLUMNS.
(See facing page.)



Nomogram for the Design of Axially-loaded Columns.

By J. C. STEEDMAN.

THE nomogram on page 242 is an aid to the design of columns of a given cross-sectional area under the action of a known axial force. The percentage of reinforcement required for any normal combination of permissible stresses in the steel and concrete can be read directly from the chart.

EXAMPLE.—Determine the reinforcement required in a short column 2 ft. square subjected to an axial load of 300 tons if the permissible compressive stresses are 760 lb. in the concrete and 16,000 lb. per square inch in the reinforcement.

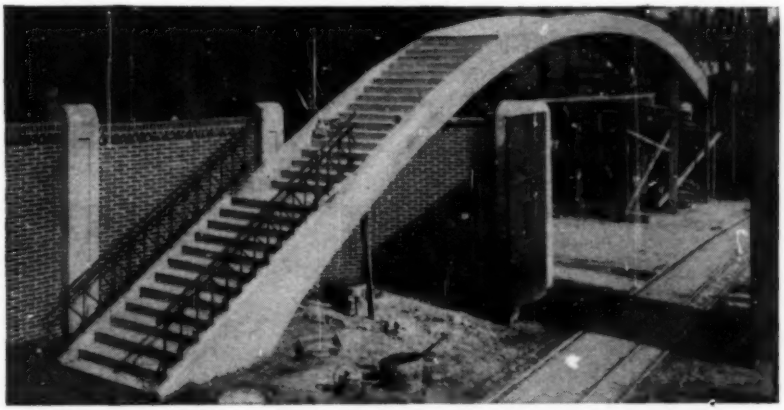
$$\frac{\text{Load}}{\text{Cross-sectional area of column}} = \frac{300 \times 2240}{576} = 1167 \text{ lb. per square inch.}$$

The broken line drawn from 1167 on the scale on the left of the chart through the intersection of the lines representing the permissible stresses in the steel and concrete meets the scale representing the percentage of reinforcement at 2·67 per cent., which is within the recommended range (0·8 to 8 per cent.). The area of steel required is $2{\cdot}67 \times 576 \div 100 = 15{\cdot}37$ sq. in., which is provided by sixteen $1\frac{1}{8}$-in. bars.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", June, 1910.

Reinforced Concrete Footbridge at Kiel.



"The bridge in our illustration was built to connect docks which have lately been erected in Kiel. The span of the (three-hinge) arch is 27 metres (88 ft.), with a rise of 5 metres (16 ft.). The working load was taken at 400 kilog. per sq. metre (82 lb. per square foot) and the exact stresses were calculated according to the latest regulations of the Prussian Ministry of Public Works. As the work was not one of great magnitude, the concrete was mixed by hand in the proportion of 1 : 5 for the arch and 1 : 6 for the abutments. The steps are made of plain concrete, and were formed direct into the arch."

Precast Panel Walls for a Church.

RECONSTRUCTION of the church of St. Julien, in Caen, France, which was destroyed during the war, was completed in 1958. The church (*Figs.* 1 and 2) is elliptical on plan with a major axis of 180 ft. and a minor axis of 90 ft., and has an elliptical dome with a rise of 16 ft. (*Fig.* 3).

The structure is supported on 172 piles, which were cast in place and each of which is of 60 tons capacity, extending to a depth of about 30 ft. where they bear on rock. The piles around the perimeter support columns 61 ft. high and 8 in. wide and varying in thickness from about 5 ft. to 1 ft. 3 in. The columns are at about

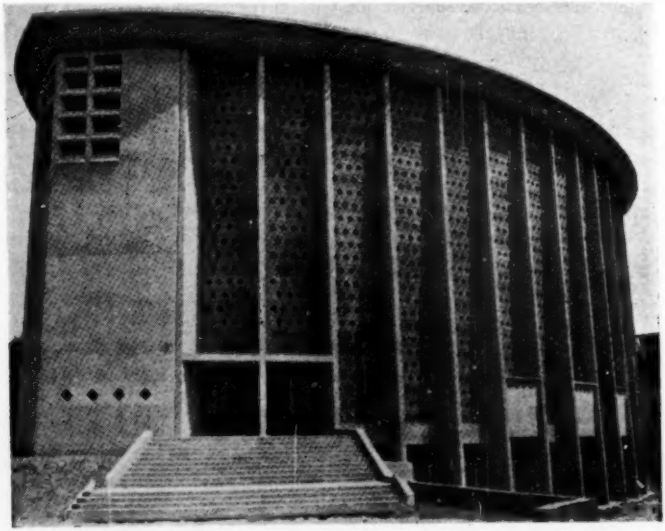


Fig. 1.—Main Entrance. (The Bells are behind the Louvres.)

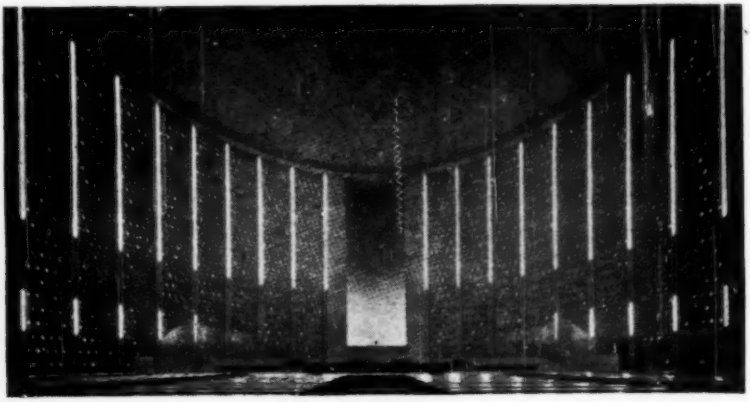


Fig. 2.—Interior.

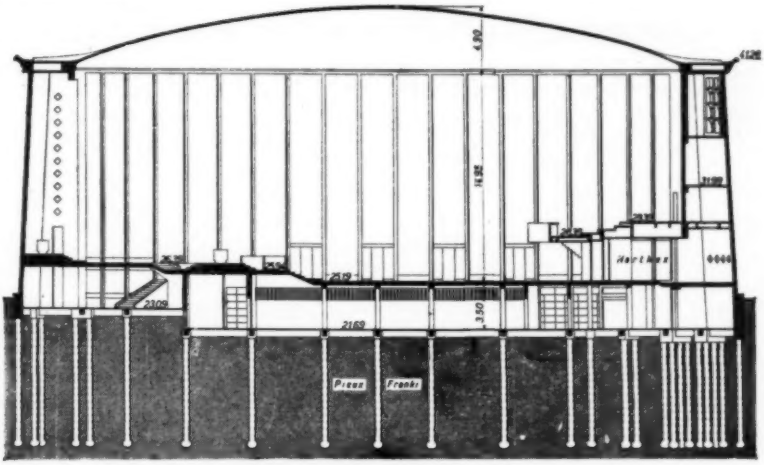


Fig. 3.—Longitudinal Section.

10-ft. centres and support the roof, the horizontal thrusts from which are resisted by a circumferential beam carried on the tops of the columns and which also forms a gutter. The basement floor and the ground floor are of beam-and-slab construction and are also supported on piles.

The slabs (*Fig.* 1) between the columns are retained in grooves formed for the full height of the columns. These slabs, which are about 9 ft. 6 in. long, 1 ft. 9 in. high, and 2 in. thick, are of two types each containing either four or five diamond-shaped windows. Along the top and bottom edges of each slab are grooves in which a bituminous material is placed to seal the joints. The slabs, which were precast on the site, were hoisted (*Fig.* 4) by a tower crane travelling on rails extending along the major axis of the building at basement level.

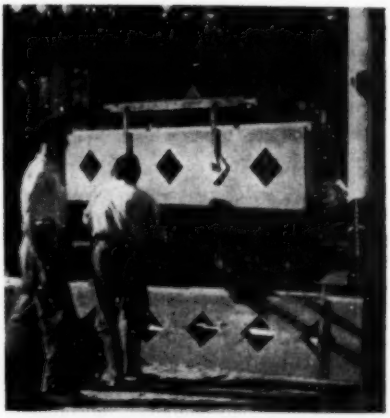


Fig. 4.—Hoisting a Wall Slab.

After completion of the walls the dome, which is 3½ in. thick, was cast on centering supported on scaffolds. The removal of the centering was controlled by means of forty hydraulic jacks, which were interconnected to ensure uniform movement of the centering. The calculated deflection of the centre of the dome, which has a rise of 16 ft., was 20 mm. but the actual deflection was only 2·5 mm. The dome is covered with two layers of bituminous felt, the upper layer having a thin aluminium sheet bonded to it during manufacture. For acoustical reasons the soffit of the dome is finished with a vermiculite plaster.

The altar is of concrete, with aggregate of Carrara marble. Except for the dome all the concrete was cast in wooden moulds lined with a plastic material and was not treated after removal of the moulds.

The reinforced concrete was designed by Société B.E.C.S.E. and the contractors were Société T.H.E.G. The foregoing notes and the accompanying illustrations are from "La Technique des Travaux" for March–April, 1959.

Removing Water from a Clay Foundation.

An earthen dam with a concrete cut-off and central puddle-clay wall was constructed at Selset, Co. Durham, for the Tees Valley Water Board. The dam is 3200 ft. long by 130 ft. high and comprises over 2,000,000 cu. yd. of earth. Part of the valley under the dam consists of 7 ft. to 10 ft. of gravel over 15 ft. to 20 ft. of soft boulder clay which in turn overlies hard boulder clay. The wet clay was incapable of supporting the dam and, after considering the removal of the soft clay and gravel and replacing it with other material, it was decided to consolidate and strengthen the soft clay by squeezing the surplus water from it by the weight of the dam. This was cheaper than replacing it with other material, and also saved a year of construction time.

The rate of increase of strength of the clay depended on the rate of consolidation, and this depended on the rate of placing of the earth and the length of the escape path for the water in the clay. To hasten the escape of the water vertical sand-wells were formed extending from the bottom of the soft clay to the surface. To ensure freedom to escape of the water forced to the surface, a layer of rubble 12 in. thick was placed on top of the gravel. The wells are 1 ft. 6 in. diameter on a grid 10 ft. square and filled with ¼-in. crushed whinstone. The longest release path for the water in the clay is therefore the distance to the nearest well (about 7 ft.). The water rises through the wells and flows away through the gravel or rubble.

In deciding the method of forming the wells it had to be considered that boulders, possibly up to 2 cu. yd. in volume, might be encountered and that running sand was known to exist in the gravel and the soft clay. Drilling was ruled out on account of the length of time required to sink the wells by this method. The method adopted was to use standard cast-insitu piling equipment and steel tubes with a thickened shoe, since it was calculated that the shearing strength of the wet clay was such that boulders encountered by the shoe would be pushed away. It was also known that the overlying gravel was loose and that, although it contained boulders, it would offer little resistance to penetration.

For average depths of clay, a steel casing 1 ft. 6 in. diameter by 30 ft. long was slipped over a mandrel of the same length. This assembly was placed on the shoe and the top of the mandrel fastened in the leads of a 65-ft. portable piling frame and driven with a 2½-tons drop hammer to the set which indicated that the shoe had reached the hard clay. The mandrel was then withdrawn, the casing was filled to ground-level with crushed whinstone, and the casing was withdrawn by a steam extractor on a crane so that the shoe was left at the bottom of the well. It was found that where a well deeper than 30 ft. was required a 45-ft. mandrel could be driven to the required depth through the 30-ft. casing, as no casing was required below the upper layers. The presence of boulders caused no difficulty.

The consulting engineers to the Board are Messrs. Edward Sandeman, Kennard & Partners, and Professor A. W. Skempton and Dr. A. W. Bishop (of the City and Guilds Engineering College) were consulted on the problem. The contractors are Messrs. Richard Costain, Ltd., and the plant for forming the sand wells was supplied by the British Steel Piling Co., Ltd.

High Temperature in a Prestressed Concrete Nuclear Reactor.

The nuclear reactor installed at Marcoule for the Commissariat à l'Energie Atomique (of France) comprises a pressure vessel of prestressed concrete and is believed to be the first of its kind constructed by this method. The temperature of the coolant, in this case carbon-dioxide gas, is generally high and means were taken in the design to ensure that the concrete did not become overheated. In operation at full power, however, the temperature of the concrete in the upper part of the vessel exceeded the temperature expected by 50 deg. C. Consequently greater stresses were induced in the concrete due to local differences of temperature. It is reported that the prestressing cables were considered to have sufficient margin of strength to withstand safely the increased forces. When the reactor was shut down temporarily for routine maintenance work to be carried out recently, it was found that the more severe conditions had caused no defect in the concrete vessel. The reactor is again working at full power but, owing to modifications now made, it is not expected that the temperatures will exceed those intended originally.

Prestressed Concrete in a Grandstand in Jamaica.

THE description in the following of the grandstand for the new racecourse at Caymanas Park near Kingston, Jamaica, is contributed by MR. J. LEVY, A.M.I. STRUCT. E., who supervised the design.

A feature of the grandstand (*Fig.* 1) is the roof which is a folded-plate reinforced concrete slab 5 in. thick spanning between prestressed beams cantilevering 50 ft. The structure is 285 ft. long and comprises two parts, one of three bays of 30 ft. each and one of six bays of 30 ft. each, with a passage 15 ft. wide between the two parts.

Fig. 1.

Design.

The depth of the folded-plate roof is 7 in. at the front edge and 5 ft. at the main columns. The slab is designed as three-pin arches spanning 30 ft. between the cantilevers and carrying an imposed load of 30 lb. per square foot. The horizontal thrust at the springing varies from 9 tons per foot at the front edge to ¾ ton per foot at the main column, and is resisted by transverse prestressing cables suitably spaced along the cantilevers.

Each cantilever beam is of I-section and was precast in parts at a factory; the parts were assembled on the site, and prestressed by means of twenty cables each of which contain twelve 0·2-in. wires. Anchorages for cables not extending the full length of the cantilevers are accommodated in flanges at the end of each precast block. The ends of the blocks next to the main column rest on the column and the next two blocks in each beam are stepped (*Fig.* 2) at the ends to assist in resisting shearing forces. Each cantilever imposes a load of up to 220 tons on the main column and an uplift of up to 80 tons on the rear column to which it is anchored by means of three twelve-wire cables extending down the column to below the level of the first floor.

The main frames, the shape of which is seen in *Fig.* 1, are of ordinary reinforced concrete design. Precast prestressed beams, fabricated at the factory by the long-line process, span at 3-ft. centres between the sloping members of the main frame. Precast slabs 3 in. thick provide the seating space and span between these beams, resting on the top of one beam and on the nib of the next beam.

Floors are provided at two levels at the rear of the stand and are formed of precast prestressed beams of I-section 12 in. deep which are laid side by side, spanning between the main frames, and covered with a screed.

Each main column transmits a load of up to 300 tons to its foundation which comprises a pile-cap, 10 ft. square and 2 ft. thick, on nine precast piles which extend about 30 ft. through strata of sand and silt to a firm bearing stratum. Each pile, which is capable of supporting 35 tons safely, is 12 in. square and is prestressed to withstand forces due to handling. One pile is provided under the rear column to resist either tension or

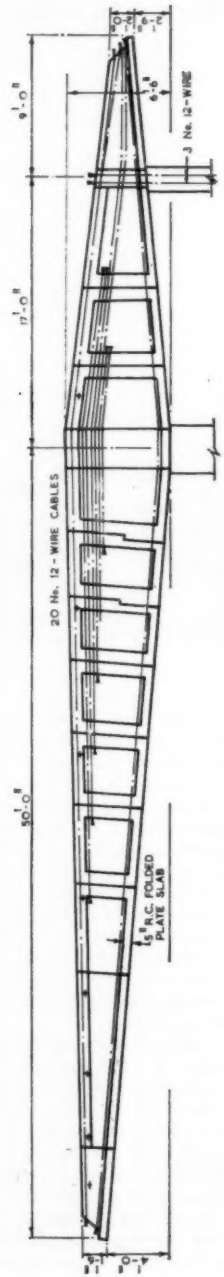


Fig. 2.—Arrangement of the Cantilever Beams.

compression, but theoretically there is no load on the foundation of this column under ordinary conditions.

Construction.

The piles were cast at a factory on four long-line prestressing beds each 250 ft. long. The prestress was transferred about seven days after casting when the concrete, which was of 1 : 1½ : 3 mixture, had attained a strength of about 4500 lb. per square inch. About thirty-six piles were produced weekly. A total of 130 piles were driven at an average rate of four or five daily by a single-acting hammer working on hanging leaders attached to a 20-ton crane on continuous tracks.

While the piles were being driven the rest of the precast members were cast and the shuttering for the roof was fabricated. The roof was constructed three bays at a time. First the cantilevered beams were assembled in position and four cables were tensioned in each beam to support the dead weight and compress the dry-packed joints between the precast parts. The roof slab was then cast to form a composite construction with the beams. When the slab had attained a strength of 3000 lb. per square inch, the transverse and the remaining longitudinal cables were tensioned. This operation raised the roof slab from the shuttering by amounts up to ¾ in., enabling the shuttering to be removed easily for erection in the next three bays. The piling and construction were completed in about thirteen months.

The architect is Mr. B. V. Oliver. The contractors and designers were Caribbean Construction Co., Ltd. The precast members were manufactured by Jamaica Concrete Products, Ltd., and prestressing was by means of P.S.C.-Freyssinet equipment.

A Retirement.

AFTER twenty-five years' practice as a consulting engineer, Mr. Leslie Turner has retired from the firm of Messrs. Leslie Turner & Partners. He will, however, continue his association with the firm in an advisory capacity.

International Association of Bridge and Structural Engineering.

VOLUMES 18 and 19* of the Publications of the International Association of Bridge and Structural Engineering have now been published.

Volume 18 contains thirteen papers of which nine are in English; summaries of some of the papers are as follows.

"BENDING OF A SECTORIAL PLATE." By Tadahiko Kawai (U.S.A.). In the English language.—Green's function for the deflection of a plate having the shape of a sector of a circle and freely supported along the radial edges and fixed along the circumferential edge is obtained in the form of double and single series. Using the single series and assuming the limit to be the radius of the sector equal to infinity, Green's functions for the bending and twisting moments on a wedge-shaped plate are derived. Moment-influence surfaces and moment surfaces are developed from this solution.

"CALCULATION OF PRESTRESSED CONCRETE MEMBERS ACCORDING TO THE STRAIN THEORY." By K. Kordina (Germany). In the German language.—The hypotheses of a strain theory of the second order are dealt with briefly together with the known approximate solutions for the determination of the additional moments from the deformation of the system. The author explains when this theory may be applied. Contrary to a conception put forward elsewhere, it is claimed that in prestressed concrete structures the strain theory applies only in exceptional cases. The plastic deformations of the concrete do not justify the application of the theory of the second order. The effect of prestressing upon the state of stress and deformation is explained with reference to characteristic structures. An actual structure was examined and the design calculations made anew. The author demonstrates that the deformations due to plastic flow which were observed in the structure may be explained in a simple manner and do not arise from the additional moments indicated by the theory of the second order.

"THE STABILITY OF RIB-REINFORCED CYLINDRICAL SHELLS UNDER LATERAL PRESSURE." By Johannes Moe (Norway). In the English language.—Formulæ are derived and graphs presented which show the stiffening effect of circumferential ribs on a complete cylinder with thin walls subjected to an external lateral pressure. As a complete cylinder generally buckles into several circumferential waves, the formulæ are thought to apply in predicting the safety against buckling near the crown of a cylindrical "shell" roof. Account must be taken of the influence of additional axial stresses on the stability of the shell.

"CYCLIC LOADING OF PORTAL FRAMES." By B. G. Neal (Wales) and P. S. Symonds (U.S.A.). In the English language.—The growth of deflections of rectangular frames under repeated cyclic loading due to the formation and rotation of plastic hinges at various stages of each loading cycle are considered. Calculations are used to show that if the intensity of load exceeds a critical value it is possible for large deflections to be produced by repeated cyclic loading, even though the intensity of the load is less than that required to cause plastic collapse. The deflections would be limited in practice only by such effects as strain-hardening. Five tests on model rectangular steel frames were carried out to confirm the theoretical predictions, and two plastic-collapse tests on similar frames were made for comparison. It is concluded that the agreement between the experimental results and theoretical predictions is satisfactory.

"ANALYSIS OF THIN ELASTIC SHALLOW SEGMENTAL SHELLS." By G.-A. Oravas (Canada). In the English language.—A method of stress analysis of thin elastic shells that are meridional segments of rotational surfaces generated by shallow segments of second-degree curves is given. The analysis includes the interaction between transverse bending and stress resultants of the shell. The solution is expressed by a trigonometrical series coupled with Kelvin functions of the mth order. The usefulness of the analysis is limited to segmental shells the circumferential angles θ of which conform to the

* Zürich: Verlag Leeman. Price 40 Swiss francs for each volume.

restriction that $\frac{\pi}{\theta}$ is an integer. Numerical results are also given for an extremely shallow segmental shell subjected to a uniformly-distributed load.

"A BRIEF REVIEW OF ANCIENT AND MODERN BUILDINGS IN TURKEY." By M. Tokoöz (Turkey). In the French language.—Some modern reinforced concrete bridges, silos, and buildings in Turkey are described and illustrated.

"AERODYNAMIC STUDY OF THE FRENCH PAVILION AT THE BRUSSELS INTERNATIONAL EXHIBITION, 1958." By P. Vallée, R. Pris and C. Bonvalet (France). In the French language.—The roof of this steel building is a series of hyperbolic paraboloids and, because of the novelty of the design, the calculations were checked by means of an aerodynamic test on a model to investigate the stability and local stresses.

The 19th Volume of "Publications" contains fifteen papers of which twelve are in English, one in French and two in German. Some of the papers included in this volume have been submitted as contributions to the 6th Congress which is to be held in Stockholm in 1960. Summaries of each paper are given in English, French and German. The contents of this volume include the following.

"Theory of Trusses." By S. O. Asplund. (In English.)

"Secondary Moments, End Rotations, Inflection Points and Elastic Buckling Loads of Truss Members." By Kuang-Han Chu. (In English.)

"The Continuous Vierendeel Girder." By I. A. El-Demirdash. (In German.)

"Application of Donnell's Theory of Circular Cylindrical Shells to the Analysis of Curved Edge Disturbances." By Ivar Holand. (In English.)

"The Design of Frameworks to Give Specific Deflections." By L. G. Jaeger and A. H. Chilver. (In English.)

"Vibrations of Bridges with Continuous Main Girders." By Vladimir Kolousek. (In English.)

"Theoretical and Experimental Researches on Continuous Box Girder Bridges." By Ichiro Konishi, Sadao Komatsu and Yushi Fukumoto. (In English.)

"A Criticism of the Plastic-Hinge Concept Used in the Plastic Theory of Structures." By B. O. Kuzmanovic. (In English.)

"Bending of Partially Loaded Simply Supported Cylindrical Shells." By S. L. Lee. (In English.)

"The Effects of Edge-Stiffening and Eccentric Transverse Prestress in Bridges." By G. Little and R. E. Rowe. (In English.)

"Orthotropic Plates and Orthotropic Cylindrical Shells with Asymmetric Ribs." By Ch. Massonet. (In French.)

"On the Analyses of a Skew Girder Bridge by the Theory of Orthotropic Parallelogram Plates." By Masao Naruoka and Hiroshi Ohmura. (In English.)

"Deformation of Ring Girders Stiffening Thin Shells of Rotation." By Gunhard-Aestius Oravas. (In English.)

"On the Relaxation of Steel Wires." By Fritz Stussi. (In German.)

"On Boundary Conditions in the Bending of Thin Elastic Plates." By A. Werfel. (In English.)

Blackwall Tunnel.

THE new approaches to the road tunnel under the River Thames at Blackwall were described in this journal for May 1960. The contract for the new tunnel, which will duplicate the old tunnel, has been awarded to Messrs. Balfour, Beatty & Co., Ltd. Two pilot tunnels of 7-ft. diameter will be driven first, in positions corresponding to the top and bottom of the main tunnel. It is intended to consolidate by chemical means the ground between the pilot tunnels before commencing to drive the main tunnel, the diameter of which will be 28 ft. 2 in. The entire work will be carried out in compressed air. About 80,000 cu. yd. of soil have to be removed. Concrete caissons will be sunk on each bank of the river and driving the tunnel will proceed from each caisson towards the centre of the river.

Change of Address.

MESSRS. L. G. Mouchel & Partners, consulting civil engineers, after having been at 38 Victoria Street, S.W.1., for about sixty years, are now at Southbank House, Black Prince Road, Albert Embankment, London, S.E.11; telephone Reliance 7691.

Standards of the American Society for Testing Materials.

THE 1958 edition of the triennial publication in which are collected the Standards issued by the American Society for Testing Materials is in ten parts. The contents of those parts* dealing with materials for concrete and reinforcement, and the dates of the latest revisions, are given in the following. Other specifications and methods of testing are given in Part 5 and relate to fire-resistance of building materials, thermal and acoustical insulating materials, and the strength of various forms of building construction. The supplements† to the collected standards were published recently and are also in ten parts. They include all revisions made up to the end of 1959. The revised standards are indicated in the following.

REINFORCEMENT.—In Part 1 are given the tentative standards for billet-steel bars (1958), rail-steel bars (1957), cold drawn steel wire (1958), axle-steel bars (1957), welded steel-wire fabric (1958), deformations of deformed steel bars (1956), large deformed billet-steel bars (1958), and high-strength billet-steel bars. The standard for fabricated steel bar or rod mats (1937) is also given. Particulars of the methods of determining the physical properties in accordance with the requirements of the foregoing standards are given in Part 3.

The tentative standard for rail-steel bars was revised in 1959, and new tentative standards were issued for high-tensile billet-steel bars and deformed billet-steel bars having a yield stress of 60,000 lb. per square inch.

PRESTRESSING STEEL.—A tentative standard (A 416–59T) applies to uncoated seven-wire stress-relieved strands for prestressed concrete and particulars are given in Part 1. The sizes are $\frac{1}{4}$ in., $\frac{5}{16}$ in., $\frac{3}{8}$ in., $\frac{7}{16}$ in. and $\frac{1}{2}$ in. diameter. The pitch of the helical wires must be not less than twelve and not more than sixteen times the diameter of the strand. The load to produce an extension of 1 per cent. must be not less than 7650 lb. for $\frac{1}{4}$-in. strand and 30,600 lb. for $\frac{1}{2}$-in. strand, the loads for intermediate sizes being proportional to the nominal cross-sectional area. The tensile strength must be not less than 9000 lb. for $\frac{1}{4}$-in. strand, 36,000 lb. for $\frac{1}{2}$-in. strand, and proportionally for other sizes. The elongation at failure must be not less than 3·5 per cent.

A new standard for uncoated stress-relieved wire was issued in 1959.

CEMENT.—Standards are given in Part 4 for Portland cement and air-entraining Portland cement and three types of special cement. Tentative standards for two types of cement and for air-entraining agents are also given. Thirteen standards and eleven tentative standards describe the methods of testing cement. Most of these standards were revised in 1958. The specifications and methods of testing cement are also issued in a separate publication entitled "A.S.T.M. Standards on Cement, 1958" (price 3.50 dollars).

Some minor revisions were made in 1959 to some of these standards and a new tentative standard for determining the fineness by means of a No. 325 sieve is included.

AGGREGATES.—Standards for aggregates (1957), and tentative standards for lightweight aggregates for blocks, insulating concrete, and structural concrete are given in Part 4. Other standards describe methods of testing aggregates and define the terms relating to these materials. Revisions to the principal standards for aggregates and to the standards for lightweight aggregates for blocks and structural concrete were made in 1959. Some methods of testing were also revised.

CONCRETE.—A standard for ready-mixed concrete (1958) is given in Part 4, in which are also tentative standards for air-entraining agents (1958), fly-ash (1957), and pozzolanas (1958). Other standards relate to calcium chloride (1956), hot-poured joint-filling material (1952), preformed joint-fillers (1949 for non-extruding types and 1953 for bituminous types), sodium chloride (1958),

* "1958 Book of A.S.T.M. Standards." Part 1, Ferrous Metals (price 12 dollars); Part 3, Metals: Test Methods (price 10 dollars); Part 4, Cement, Concrete, Mortars, Road Materials, Waterproofing, Soils (price 12 dollars); Part 5, Masonry Products, Ceramics, Thermal Insulation, Sandwich and Building Constructions, Acoustical Materials, Fire Tests (price 12 dollars). Parts 6 to 10 deal with wood, paper, and other materials, petroleum products, paint, fuels, plastics, rubber, textiles, soap, etc. Obtainable from the Society at 1916 Race Street, Philadelphia, U.S.A.

† "1959 Supplement to the Book of Standards." In ten parts; price 4 dollars each part. Obtainable from the A.S.T.M. at the address given above.

membrane curing compounds (1958), and waterproofed paper (1958). Thirty-five standard or tentative methods of testing concrete and some of the foregoing materials are also described.

Several of the methods of testing concrete were revised in 1959 and a new tentative standard relating to the effectiveness of admixtures in preventing excessive expansion due to alkali-aggregate reaction was published.

CONCRETE PRODUCTS.—Standards are given in Part 5 for load-bearing and non-load-bearing solid and hollow blocks (1952), and for gulleys and manholes (1939), bricks (1955), drain tiles (1958), and sewer pipes (1958). One standard and three tentative methods of testing concrete products are also given. Minor revisions were made to some of these standards in 1959.

Concrete Materials.

The revised edition of "A.S.T.M. Standards on Mineral Aggregates and Concrete" (obtainable from the American Society for Testing Materials; price $4.75) includes about one hundred standards relating to aggregates and concrete, with some standards for other materials associated with concrete roads. Most of the standards have been revised since the previous issue in 1954. New standards include those for blastfurnace-slag cement; testing fresh concrete by ball penetration; dry materials for mortar and concrete supplied mixed in packets; raw and calcined natural pozzolana for use in concrete; testing the rate of hardening of mortars screened from concrete; aggregates for masonry grout; and testing the resistance of concrete to abrasion.

A Distributor for Concrete and Aggregate.

Fig. 1.

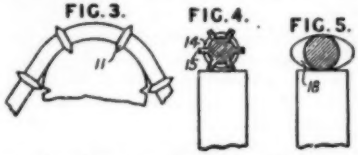
THE purpose of the machine shown in *Fig.* 1 is to receive concrete or aggregate from an overhead mixer or store and transfer it to a belt-conveyor for transportation to the part of the site where it is required. The machine comprises a hopper of 1¼ cu. yd. capacity and a conveyor 3 ft. wide which passes beneath the hopper at speeds up to 100 ft. per minute and is driven by a 2 h.p. electric motor. The flow of material from the hopper to the belt is controlled by an adjustable outlet, and up to 35 cu. yd. of material can be discharged hourly on to a distributing conveyor. Local or remote controls are available. The machine is mounted on a pair of wheels, but when in operation it is supported on two screw-jacks and a hinged support at the rear of the machine.

The distributor is supplied by the Wickham Engineering Co., Ltd.

Patent Applications.

Deformed Reinforcement.

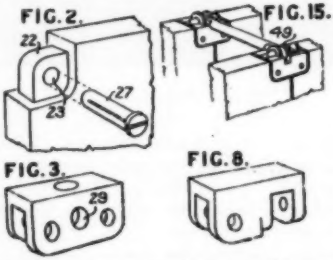
STEEL rods or wires for reinforced concrete are provided with transverse ridges



(11, *Fig.* 3) or longitudinal and transverse ridges (15) and (14) respectively (*Fig.* 4) or transverse ridges (18) projecting only on two opposite sides of the rod (*Fig.* 5).—No. 818,854. W. Hufnagl. August 26, 1957.

Shuttering.

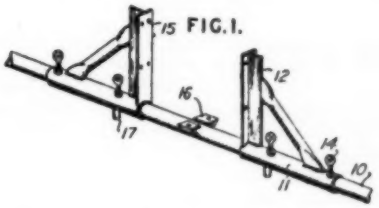
SHUTTERING for casting concrete comprises panels having the corners cut away so as to leave a rib (22) with a hole (23) for a dowel (27) which secures a tie block (*Fig.* 3) adapted to connect two panels together at the corners, the block having a hole (29) for a tie bolt connecting shuttering on opposite sides of a cavity in which a wall is to be cast. Another form of block connects four panels and a still further form



(*Fig.* 8) connects panels at right angles. The blocks may have pivots (49, *Fig.* 15) for the tie bolts so that the panels can be swung on the end of the tie bolts away from the cast concrete. Two panels may be connected at right angles by means of an integral extension of a tie block such as that shown in *Fig.* 3 connected by a dowel to another similar tie block. Shuttering is shown for casting columns and beams.—No. 819,657. A. Fioruzzi. April 19, 1956.

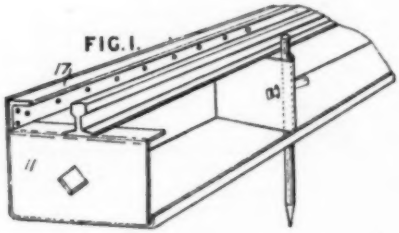
Clamps for Beam Shuttering.

A CLAMP for securing beam shuttering to scaffolding comprises a tube 10 on which slides a pair of sleeves 11, having jaws 12 for clamping the shuttering, which can be secured to the tube by screws 14, the tube being securable to tubular scaffolding by spigots 17 depending from the clamps or the tube 10. Alternatively brackets may be welded to the sleeves 11 provided with holes for



bolting to end plates on the scaffolding. Holes 15 are provided in the T-section jaw 12 for bolts to secure the shuttering, the base of which rests on plates 16. Vertical extensions may be fitted to the jaws 12.—No. 820,326. Kwikform, Ltd. July 23, 1957.

Road Forms.

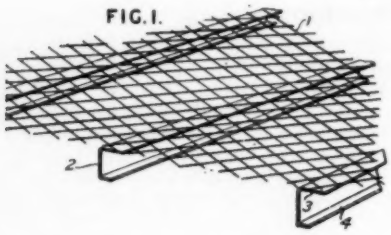


THE upper edge of the moulding flange 11 of a road form is sharp and square-cut and braced by a stiffening member 17 so as to leave a square edge without a lip on the cast concrete.—No. 821,508. J. Grundy & Co., Ltd. May 6, 1957.

Combined Reinforcement and Shuttering.

A COMBINED reinforcement and shuttering for suspended concrete slabs, such as floors or roofs, comprises a sheet 1 of open mesh metal having on one side channel shaped metal ribs 2, each having one flange 3 wider than the other 4, the wider

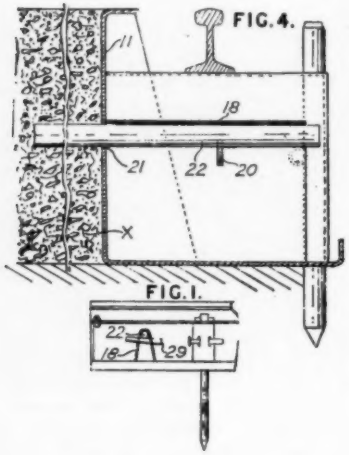
FIG. 1.



flange 3 being welded to the sheet 1. The sheet 1 may be of expanded metal. In forming a ceiling, the material is laid between supporting walls or other members with the ribs 2 above the sheet 1 and concrete is poured so that the sheet 1 acts as a shuttering and the ribs 2 reinforce the concrete. Finally the undersurface of the sheet is plastered.—No. 822,005. Expanded Metal Co., Ltd. April 26, 1956.

Road Shuttering.

FIG. 4.



A ROAD shuttering has an aperture 21 in the moulding face 11 for a dowel bar 22 intended to be left in the concrete X and held by a wedge 20 in the apex of an inverted U-plate 18 welded to the road shuttering or the dowel rests in a tube in alignment with the aperture 21. The other end of the dowel is intended to be embedded in the next bay of concrete to form an expansion joint.—No. 821,509. J. Grundy & Co., Ltd. May 16, 1957.

Rubber Bearings for a Bridge.

THE Methley bridge, which carries the road between Leeds and Barnsdale over the River Calder in the West Riding of Yorkshire, has three equal spans of 70 ft. The bridge comprises concrete beams, which are prestressed longitudinally and transversely by means of steel wires, and are supported by reinforced counterforted abutments on the banks and plain concrete piers in the river. The ends of the beams rest on rubber and metal bearings

Fig. 1.

(*Fig.* 1). The loads imposed on each bearing are up to 63,650 lb. vertically and 4660 lb. horizontally. The bearings enable the beams to move longitudinally 0·129 in. in either direction. The greatest rotation due to deflection is 0·005 radians. The bearings are formed of a rubber compound retained between metal plates and containing material which prevents oxidation and ozonation of the rubber and corrosion of the metal. No fixing to the piers or beams is provided. These bearings are claimed to fulfil the same function as rollers, rockers, or sliding plates but to require no lubrication or maintenance.

The bridge was designed by Mr. S. M. Lovell, O.B.E., County Engineer and Surveyor of the West Riding of Yorkshire. The contractors are Messrs. J. L. Kier & Co., (London) Ltd. The bearings were supplied by Metalastik, Ltd.